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12-14 September, 2016
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**Room Location Key:**

- KH-LS = Kulturens Hus, Lilla Salen
- KH-OB = Kulturens Hus, Olga Bardh

## SUNDAY 11.9.2016

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| 7:30 – 8:00 | Registration open  
Author’s Coffee (KH – Lounge) |
| 8:15 – 9:30 | **Opening Session (KH – LS)**  
8:15 Welcome and opening remarks – Prof. Erling Nordlund  
8:25 LTU Vice-Chancellor Johan Sterte  
8:35 Rut och Sten Prof. Erling Nordlund  
8:45 STRIM – Dr. Jenny Greberg  
9:00 **Opening Keynote:** Innovative and controversial support for rockbursting conditions, Prof. Dick Stacey |
| 9:30 – 10:00 | Coffee break |
| 10:00 – 11:20 | **Session 1: Numerical Modeling 1 – Support (KH - LS)**  
**Session Chair:** Prof. Sunniva Haugen |
| 11:20 – 12:20 | Lunch (KH – Restaurant) |
| 12:30 – 14:00 | **Session 2: Seismic/Dynamic Issues 1- Rock Mass (KH – LS)**  
**Session Chair:** Prof. Bruce Hebblewhite |
| 14:00 – 14:30 | Coffee break |
| 14:30 – 15:10 | **Sponsor Session (KH – LS)**  
**Session Chair:** Dr. Tristan Jones |
15:15 – 16:45  
**Session 3: Ground Support Testing 1 – Rebar Bolts (KH – LS)**  
*Session Chair: Dr. Johan Wesseloo*

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**TUESDAY 13.9.2016**

7:30 – 8:00  
**Author’s Coffee (KH – Lounge)**

8:00 – 9:30  
**Session 4: Ground Support Testing 2 – Panels and Cables (KH – LS)**  
*Session Chair: Ms. Lauriane Bouzeran*

8:30 – 9:30  
**Session 5: Seismic/Dynamic Issues 2- Support (KH – OB)**  
*Session Chair: Dr. Catrin Edelbro*

9:30 – 10:00  
Coffee break

10:00 – 11:40  
**Nordic Speaker Session**  
**Session 6: Risk-Based Support Design (KH – LS)**  
*Session Chair: Dr. Charlie Li*

10:00 – 11:40  
**Session 7: Support Response (KH – OB)**  
*Session Chair: Adj. Prof. Jonny Sjöberg*

11:40 – 12:40  
Lunch (KH – Restaurant)

13:00 – 14:40  
**Nordic Speaker Session**  
**Session 8: Case Studies 2 - Tunneling (KH – LS)**  
*Session Chair: Ms. Kristina Jonsson*

13:00 – 14:40  
**Session 9: Case Studies 1 - Mining (KH – OB)**  
*Session Chair: Ms. Märit Berglind-Eriksson*

14:40 – 15:10  
Coffee break

15:10 – 16:50  
**Session 10: Support Design Applications (KH – LS)**  
*Session Chair: Dr. William Joughin*

15:10 – 16:50  
**Session 11: Numerical Modeling 2 – Ground Movement (KH – OB)**  
*Session Chair: Prof. Wacheng Zhu*

18:00 - 23:00  
Conference Dinner (KH - Restaurant)

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KH-OB = Kulturens Hus, Olga Bardh

7:30 – 8:00  Author’s Coffee (KH – Lounge)

8:00 – 9:50  Session 12: Rock Slope Stabilization (KH – LS)
  Session Chair: Prof. John Hadjigeorgiou

9:50 – 10:20  Coffee break

  Session Chair: Dr. Eva Hakami

12:10 – 13:10  Lunch (KH – Restaurant)

13:10 – 15:00  Session 14: Ground Support – Applications (KH – LS)
  Session Chair: Ms. Beatrice Lindström

15:00 – 15:15  Conference Closing

THURSDAY 15.9.2016

Technical Visits – As scheduled


Technical Visits – As Scheduled

Room Location Key:   KH-LS = Kulturens Hus, Lilla Salen   KH-OB = Kulturens Hus, Olga Bardh
Paper Presentation Schedule

MONDAY 12.9.2016

Opening Session (KH – LS)

09:00  **Keynote Speaker:** Innovative and controversial support for rockbursting conditions  
       Stacey, T.R.

Session 1: Numerical Modeling 1 – Support (KH - LS)

10:00  Advanced 3DEC bolt model for simulation of ground support performance in highly fractured and bulked rock masses  
       Bouzeran, L.; Furtney, J.; Hazzard, J.; Lemos, J.V.; Pierce, M.

10:20  Three dimensional numerical modelling of the deep ALborz tunnels with composite liners in squeezing condition  
       Molladavoodi, H.; Bazidehno, H.

10:40  Discrete element modelling of steel wire mesh and rockbolt plate  
       Xu, C.; Tannant, D.D.

11:00  Numerical modelling of dynamic response of underground openings under blasting based on field tests  
       Yi, C.P.; Nordlund, E.; Nyberg, U.; Shirzadegan, S.; Zhang, P.
**Session 2: Seismic/Dynamic Issues 1- Rock Mass (KH – LS)**

12:30  **Keynote Speaker:** Rock support in a Cut & Fill mine – from the operational perspective of a mine manager  
Haugen, S.

13:00  Failure mechanism related to accelerating creep of rock triggered by dynamic disturbance  
Zhu, W.; Li, S.; Niu, L.; Wang, Q.; Wei, J.

13:20  Integrating microseismic and 3D stress monitoring with numerical modeling to improve ground hazard assessment  
Tonnellier, A.; Bigarré, P.; Bouffier, C.; Fjellström, P.; Mozaffari, S.; Nyström, A.; Renaud, V.

13:40  In-situ dynamic testing of rock support at LKAB Kiirunavaara mine  
Shirzadegan, S.; Nordlund, E.; Zhang, P.

**Session 3: Ground Support Testing 1 – Rebar Bolts (KH – LS)**

15:15  **Keynote Speaker:** Rock Support : Degradation and Failure  
Hadjigeorgiou, J.

15:45  Strain of steel rebar vs rock bolt elongation on a laboratory stand  
Korzeniowski, W.B.; Herezy, Ł.; Skrzypkowski, K.

16:05  Critical embedment length of fully grouted rebar bolts  
Li, C.C.; Høien, A.H.; Kristjansson, G.

16:25  An Optical sensor for capturing the three-dimensional bending of bolts  
Forbes, B.; Hyett, A.J.; Vlachopoulos, N.

Session 4: Ground Support Testing 2 – Panels and Cables (KH – LS)

08:00  **Keynote Speaker:** Cost increase, causes and improvements for underground roads and railway  
       Lundman, P.

08:30  A follow up to the behaviour of cable bolts in shear; Experimental study and mathematical modelling  
       Aziz, N.; Mirza, A.; Nemcik, J.

08:50  Parametric study of cable bolt performance under axial loading in medium strength synthetic rock  
       Li, D.; Hagan, P.C.; Saydam, S.

09:10  Drop testing of concrete panels and welded wire mesh at LKABs Kiirunavaara mine  
       Swedberg, E.; Krutrök, B.

Session 5: Seismic/Dynamic Issues 2- Support (KH – OB)

08:30  Scattering of SH-waves by a shallow circular lined tunnel with an imperfect interface  
       Yi, C.P.; Johansson, D.; Nyberg, U.

08:50  Establishment of experimental sites in three Swedish mines to monitor the in-situ performance of ground support systems associated with mining-induced seismicity  

09:10  Local seismic systems for study of the effect of seismic waves on rock mass and ground support in Swedish underground mines (Zinkgruvan, Garpenberg, Kiruna)  
**Nordic Speaker Session - Session 6: Risk-Based Support Design (KH – LS)**

10:00   **Nordic Speaker**: Rock support in the Malmberget Mine  
         Töyrä, J.

         Lappalainen, P.J.; Lamberg, M.J.

10:40   **Nordic Speaker**: Rock support in the Boliden mines  
         Marklund, P.; Sandström, D.; Nyström, A.

11:00   Real-time risk assessment and ground support optimisation in underground mines  
         K. Mishra, R.; Janiszewski, M.; Ritala, F.; Siren, T.; Uotinen, L.

11:20   A risk-based approach to ground support design  
         Joughin, W.C.; Mpunzi, P.; Muaka, J.J.M.; Sewnun, D.; Wesseloo, J.

10:00 – 11:40  **Session 7: Support Response (KH – OB)**

10:00   Comparison between dynamic ground support methods (dynamic bolting)  
         Näsi, J.; Harju, H.

10:20   Methodology for stability analysis of entry-type underground excavations  
         Burgos, L.; Delonca, A.; Vallejos, J.A.

10:40   Sensor techniques to monitor installation and status of rock bolts  
         Gustafsson, L.K.K.A.

11:00   Production-blast-induced crosscut performance in a weak biotite schist- a comparison of three high-deformation bolt types  
         Jones, T.H.

11:40   An instrumentation project to investigate the response of a ground support system to stoping induced deformation  
         Sweby, G.J.; Dight, P.M.; Gamble, N.; Potvin, Y.
### Nordic Speaker Session - Session 8: Case Studies – Tunneling (KH – LS)

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<td>Lindström, B.</td>
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<td>Nordic Speaker: Tunneling challenges in young volcanic rocks in Iceland</td>
<td>Geir, S.</td>
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<td>13:40</td>
<td>Nordic Speaker: Tunnelling methodology for difficult rock mass conditions in Deep Subsea Tunnels</td>
<td>Nilsen, B.</td>
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<td>14:00</td>
<td>Grouting and excavation support in Doha - Simple but challenging</td>
<td>Thurner, R.; Kulmer, R.; Raja, S.U.</td>
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<td>14:20</td>
<td>Tunnelling and reinforcement in heterogeneous ground - A case study</td>
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<td>Ground support practice at Glencore’s Nickel Rim South Mine - With a link to seismic monitoring data</td>
<td>Simser, B.; Butler, T.</td>
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<td>13:20</td>
<td>Mining in extreme squeezing conditions at the Henty mine</td>
<td>Roache, B.</td>
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<tr>
<td>13:40</td>
<td>Ground support for mining through weak graphitic faults at the Casa Berardi mine, Quebec</td>
<td>Armatys, M.; Board, M.P.</td>
</tr>
<tr>
<td>14:00</td>
<td>A case study review of the double fatality coal burst at Austar Colliery in NSW, Australia in April 2014</td>
<td>Hebblewhite, B.; M. Galvin, J.</td>
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<td>14:20</td>
<td>A numerical modelling case study - correlation of ground support instrumentation data with a three dimensional inelastic model</td>
<td>Sweby, G.J.; Dight, P.M.; Potvin, Y.</td>
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<td>Lubosik, Z.; Prusek, S.; Walentek, A.; Wrana, A.</td>
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<td>15:50</td>
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<td>Potvin, Y.; Hadjigeorgiou, J.</td>
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<td>16:10</td>
<td>Methodology for incorporating new elements of support for the mine design in El Teniente mine</td>
<td>Muñoz J., A.; Rojas V., E.; Zepeda A., R.</td>
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### Session 11: Numerical Modeling 2 – Ground Movement (KH – OB)

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<td>15:10</td>
<td>The contact problem of the supported circular cylindrical underground opening in elastic rock</td>
<td>Exadaktylos, G.E.; Stavropoulou, M.S.</td>
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<td>15:30</td>
<td>The use of elastic superposition as part of a multi-tired probabilistic ground support design approach</td>
<td>Wesseloo, J.</td>
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<td>15:50</td>
<td>Experimental and numerical analysis of face pressure in EPB shield in East-West lot of line 7, Tehran subway</td>
<td>Maboudi, V.; Molaei, F.; Rahimi, S.; Siavoshi, H.</td>
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**Session 12: Rock Slope Stabilization (KH – LS)**

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<tr>
<td>08:00</td>
<td><strong>Keynote Speaker:</strong> Tunnel design considering rock mass and shotcrete time-dependent properties</td>
<td>Celestino, T.B.</td>
</tr>
<tr>
<td>08:30</td>
<td>Photogrammetric calculation of JRC for rock slope support design</td>
<td>Sirkiä, J.; Iakovlev, D.; Kallio, P.; Uotinen, L.K.T.</td>
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<tr>
<td>08:50</td>
<td>Remediation of large scale structures in open pit mining</td>
<td>Bergman, A.</td>
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<td>09:10</td>
<td>A proposed workflow for slope stabilization projects</td>
<td>Botsialas, K.</td>
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<td>09:30</td>
<td>Building a structural model for assessing potential pit slope failure - a case of a Norwegian mine</td>
<td>Morales, M.; Botsialas, K.; Holmøy, K.H.; Panthi, K.K.</td>
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**Session 13: Ground Support – Theory and Advancement (KH – LS)**

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<td>10:20</td>
<td><strong>Keynote Speaker:</strong> Some aspects on ground behaviour and rock support challenges in difficult ground, from a Scandinavian perspective</td>
<td>Sturk, R.</td>
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<td>10:50</td>
<td>Dynamic twisted rockbolt for underground excavation in deep mine conditions</td>
<td>Pytel, W.; Mertuszka, P.; Szeptun, K.</td>
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<td>11:10</td>
<td>An experimental study to investigate the interaction of backfill and rock mass</td>
<td>Moser, A.; Ladinig, T.; Wagner, H.; Wallner, F.</td>
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<td>11:30</td>
<td>Laboratory Investigation of Rock-Shotcrete Deboning Due to Ice Growth using Acoustic Emission</td>
<td>Manali, G.; Dineva, S.; Nordlund, E.</td>
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<td>11:50</td>
<td>Theoretical investigation of the effect of stress on the performance of support systems based on Rock Mass Rating (RMR) support recommendations</td>
<td>Karakaplan, E.; Başarir, H.; Wesseloo, J.</td>
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<tr>
<th>Time</th>
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<td>13:15</td>
<td><strong>Keynote Speaker:</strong> Strategic use of urban underground space</td>
<td>Knights, M.</td>
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<tr>
<td>14:05</td>
<td>Scale effect of thin spray-on liners for pillar reinforcement</td>
<td>Guner, D.; Ozturk, H.</td>
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<td>14:25</td>
<td>Fibre reinforced spray concrete- Minimum performance requirement to meet safety needs</td>
<td>De Rivaz, B.</td>
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<td>14:45</td>
<td>Ground support and reinforcement system remediation at the Cethana Power Station, Tasmania, Australia</td>
<td>Hills, P.B.; Liang, M.; S. Pennington, D.; Weller, J.</td>
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Tunnel design considering rock mass and shotcrete time-dependent properties

T.B. Celestino  University of Sao Paulo and Themag Engenharia, Brazil

Abstract

The use of sprayed concrete for tunnel support involves analysing the complex phenomenon of the response of the material to increasing load while it changes from paste to a competent structural element. The mechanical properties of sprayed concrete not only depend on time, but also on its loading history. In addition to the support material, some ground masses also present time-dependent mechanical behavior. These peculiarities influence the structural design, displacement evaluation and assessment of excavation stability. While rigorous treatment of all the phenomena presented above may still lead to complex analysis procedures, not yet practical for regular design activities, their importance requires some degree of consideration. The majority of the accidents in conventional tunnel construction occur in the shady zone of consideration of excavation stability or support integrity. On the other hand, disregarding early-age sprayed concrete strength for excavation stability analysis may be uneconomical. The paper presents two simplified analysis procedures to take time-dependent properties of support and ground mass into consideration. The first one is an axisymmetric time integration procedure which was validated with instrumentation data of a sprayed concrete supported tunnel excavated through a ground mass with pronounced time-dependent mechanical behavior. The second one, more general, is a three-dimensional finite element analysis which overcomes limitations of axisymmetric approximations. Both procedures may be adopted for the analysis of the influence of the excavation advance rate on the excavation-induced displacements and internal forces in the support structure. The three-dimensional analysis can also provide information related to the excavation stability. A summary of the time dependent properties of sprayed concrete at early age is also presented.

1 Introduction

Tunnel design procedures usually consider two distinct situations to evaluate stability: at the excavation face, where no support has been installed yet and three-dimensional conditions occur; and far away from the face, where the support has already been installed and full load acts on the support structure under plane strain conditions.

When sprayed concrete is used for support, an intermediate situation takes place. It is seldom considered at the design stage due to difficulties in evaluating the intensity of the load transferred by three-dimensional conditions, and the interaction between the ground mass (sometimes time-dependent) and the highly time-dependent sprayed concrete rheology.

Most of the occurrences of accidents due to excavation instability take place in that zone. Usually, safety is evaluated by means of excavation stability methods considering young shotcrete with no strength. The length of excavation with young shotcrete is considered as unsupported for the sake of excavation stability evaluation.

This procedure, however, may lead to uneconomical and sometimes unfeasible design. The latter may occur in some cases of sequential excavation method in weak rock.

Correct and mechanically-sound procedures should consider the three-dimensional load transfer mechanism at the excavation face and interaction between support and the ground mass, both of which may present time-dependent behavior. Concrete has actually zero strength and stiffness when sprayed; mechanical properties increase with time but also depend on the loading history. In principle, this brings no serious difficulty for currently available numerical methods. However, some simplifications are usually
needed to simulate the complex rheology of young sprayed concrete which presents both time- and load-dependent properties, as well as load-independent strains. With respect to the ground mass, difficulties still exist to simulate the stand-up time which depends on its properties as well as the excavation dimensions. However, progress has occurred in both areas.

Design methods considering young sprayed concrete have been developed but are not yet practical enough to be adopted for routine design activities.

This paper presents information about time-dependent properties of sprayed concrete since early age and analysis procedures to take those into consideration by means of 3-dimensional interaction analysis. A simplified procedure of time integration for convergence confinement method with axisymmetric geometry is also presented. It is shown that with simplifications which do not seriously compromise the results it is already feasible to adopt this type of analysis for design. This must contribute to decrease the number of collapses in the most critical zones of tunnel construction.

2 Time-dependent behavior of sprayed concrete

Strains in sprayed concrete have both dependent and independent components of external load. External load independent strains are due to thermal variation and shrinkage. Load dependent strains can be instantaneous and time-deferred.

Thermal strains depend on both temperature increase and thermal expansion coefficients. Byfors (1980) compiled data on the evolution of thermal expansion coefficients. Values are higher up to 6-hour age and become constant after 20 hours. Figure 1 presents data compiled by Ferreira (2003) from the literature.

![Figure 1](image_url)

**Figure 1** Evolution of the thermal expansion coefficient for concrete (Ferreira, 2003)

Ferreira (2003) shows that results of tests available in the literature (e.g. Schubert, 1988 and Kuwajima, 1991) indicate that thermal expansion coefficients are lower for sprayed concrete than for conventional concrete.

Schubert (1988) presents total load-independent strains separated into shrinkage and those due to thermal effects. Both of them depend on the age of the concrete.
Load-dependent strains are due to both elastic and viscous phenomena. Elastic and viscous parameters are time and load dependent.

Ferreira (2003) tested sprayed concrete structural models in the field and measured both load-dependent and independent strains. Figures 2 and 3 present temperature increase and load-independent strains (in με) measured at both internal and external fibers for the first 6 hours after spraying.

Long term (28 days) load-independent strains are shown in Figure 4. The cyclic behavior of stain is due to ambient temperature variation, which cannot be avoided in field tests. It can be observed that the amplitude of stain variation at the external fiber is higher (about 30με at 5-day age) than that at the internal fiber (about 15 με at the same age). The models were 15cm thick.

Evolution of the deformation modulus is shown in Figure 5. Values for the modulus were obtained by applying increasing load to the models approximately simulating the history of loading on tunnel support. The scatter is due to different conditions of spraying on porous or impervious soil. On porous non-saturated soil, the concrete in contact with the ground suffered severe shrinkage due to water migration caused by suction. In such cases, the elastic modulus was substantially lower due to higher internal stresses at early age. Placement of an impervious membrane on the soil before spraying concrete stopped water migration and significantly decreased shrinkage. The values of the elastic modulus are significantly higher than in the case of porous soil, as much as 50%. Similar results were obtained when spraying on impervious soil.

![Figure 2](image-url)  
*Figure 2  Temperature evolution in sprayed concrete (Ferreira, 2003)*
Figure 3  Low age load-independent strains for sprayed concrete (Ferreira, 2003)

Figure 4  Long term load-independent strains (Ferreira, 2003)
3 Time-dependent behavior of ground masses

Time-dependent behavior of ground masses can be represented by rheological models involving viscous elements in combination with elastic and/or plastic elements.

For very large deformations which involve plastic mobilization of the material, it is essential that rheological models adopted should include a yielding element. Barla et al. (2008) presented the fundamentals of three models for the analysis of tunnels excavated in highly squeezing ground, some of which presenting radial displacements of more than one meter. Such large displacements also involve large strains leading to internal strength mobilization and plastic strains. The models have Mohr-Coulomb or Drucker-Prager yielding elements. The model which best described convergence of tunnels excavated in highly squeezing ground was the SHELVIP (Stress Hardening Elastic Viscous Plastic Model). It is derived from the Perzyna’s overstress theory with the addition of a time-independent Drucker Prager component.

For lower level of strain not enough to mobilize the material strength, viscoelastic models perform well. Goodman (1985) proposes the use of the Burger’s model for rock masses. Celestino (1997) observed that settlements in tunnels excavated through stiff Tertiary clays increased 30% in 4.5 years due to creep. Volumes of surface and deep settlements were always very close to each other, indicating that consolidation was not the cause of the settlement. Therefore, the assumption of settlements caused by constant-volume creep is appropriate. Fitting the Burger’s model to field displacement measurements resulted in good agreement. The numerical model described in section 4.1 adopts the Burger’s model for the time-dependent behavior of the ground mass.

When tunneling through ground masses with pronounced squeezing behavior, it is common that large radial displacements take place before hardening of the sprayed concrete, severely damaging the young support. Golser (1995) proposed spraying concrete along longitudinal zones separated by gaps which close at the initial stage of displacement while the sprayed concrete (protected of increasing tangential stresses due to the gaps) gains strength. The procedure was first successfully adopted for the Tauern Tunnel in Austria.

In order to develop some reaction by the isolated sprayed concrete zones and improving the early effectiveness of the support, lining-stress controller elements (Schubert, 1996) were developed. They consist of buckling tubes installed in the gaps.
4 Tunnel support considering young sprayed concrete

As mentioned in the Introduction, the consideration of young sprayed concrete with its time-dependent properties is needed for both support design and face stability purposes. Time-dependent considerations for the ground mass can also be implemented when needed.

Hellmich (1999), Moritz and Brandtner (2009), Gschwandtner and Galler (2012) and Golser (2001) present analysis procedures considering complex rheological models for the sprayed concrete. It will be shown that even simple elastic models considering an increasing value for the elastic modulus give very good results. The elastic modulus is reduced to take other sources of strain into consideration.

The next two sections present numerical procedures to take into consideration the time-dependent behavior of the sprayed concrete for tunnel support. Initially an axisymmetric procedure is presented, considering the time-dependent behaviors of both ground mass and sprayed concrete. A more complex 3-dimensional finite element analysis is also presented.

4.1 Axisymmetric Procedure

A procedure of time integration for axisymmetric geometry and initial stresses was presented by Celestino and Guimaraes (1994). Both ground mass and sprayed concrete were assumed to have time-dependent behavior.

The rock mass is assumed to behave as Burger’s model. Strain $\varepsilon$ and stress $p$ are related according to the following differential equation:

$$\eta_1 \ddot{\varepsilon} + G_1 \dot{\varepsilon} = \eta_2 \dot{p} + \left(1 + \frac{G_2}{G_1} \frac{\varepsilon}{\eta_1} \right) \dot{p} + \left(\frac{G_2}{\eta_2} \right) p$$

(1)

where $G_1$, $G_2$, $\eta_1$ and $\eta_2$ are the model parameters.

Three-dimensional effects of load transfer at a particular cross section as the excavation progresses are considered according to the expression proposed by Schwartz and Einstein (1980):

$$\Delta P_e = \Delta \lambda_d P_1$$

(2)

$$\lambda_d = 0.98 - 0.57 \frac{L_d}{R}$$

(3)

where $P_e$ is a fictitious external load on the support at a cross section located at a distance $L_d$ behind the face; $P_1$ is the initial stress at the tunnel axis before excavation; $R$ is the tunnel radius and $\lambda_d$ is the stress relief factor.

Figures 6 and 7 show schematically the different elements installed along the tunnel length, the longitudinal load transfer mechanism and time-dependent displacements of the excavation.
Figure 6 Load transfer mechanism due to excavation and time-dependent displacements of the ground mass

The strain increment $\varepsilon_{t_1,t_2}$ accumulated in the time interval $t_1 - t_2$ is given by:

$$\varepsilon_{t_1,t_2} = \int_{t_1}^{t_2} \frac{1}{E(t)} d\sigma,$$

(4)

The deformation modulus $E_t$ at age $t$ (days) is assumed to relate to the values at 28 days ($E_{28}$) according to the expression proposed by CEB – FIP (1978):

$$E_t = E_{28} \sqrt{\frac{42+0.85t}{t}}$$

(5)
Figure 7  Load increase at a support element due to excavation of one round
For the excavation phase, assuming that it causes a stress relief linearly dependent on time:

\[ p = at + p_0 \]  

(7)

Displacements of the ground mass can be obtained by integration of Equation 1 considering Equation 7. The result will depend on the stress relief rate \( a \) from Equation 7:

\[
 u = \frac{E}{2} \left( \frac{at^2}{2 \eta_2} + \left( \frac{1}{G_1} + \frac{1}{G_2} \right) + \frac{P_0}{\eta_2} \right) t + \left[ \frac{\eta_2}{G_1} \left( \frac{1}{G_1} + \frac{1}{G_2} \right) - \frac{P_0}{G_2} \right] \exp \left( - \frac{G_2}{G_1} t \right) + \left[ P_0 - \frac{\eta_2}{G_1} \left( \frac{1}{G_1} + \frac{1}{G_2} \right) \right]
\]  

(8)

The excavation of a round with length \( s \) during time \( t_e \) causes a stress relief which can be approximated by:

\[
 \Delta P_e = at = \frac{P_I (\lambda_0 - \lambda_{s/2})}{t_e}
\]  

(9)

Substituting the stress relief rate \( a \) from Equation 9 and \( P_0=0 \) into Equation 8, an expression for ground displacement as a function of time is obtained.

During concrete spraying, excavation does not progress and there is no load increase, i.e. \( a=0 \).

Displacements due to ground mass strain can be obtained by substituting into Equation 8:

\[
 P_0 = P_I (\lambda_0 - \lambda_{s/2})
\]  

(10)

Substituting Equation 3 into Equation 10:

\[
 P_0 = \frac{0.57 P_{Ie}}{2R}
\]  

(11)

An application to the experimental Four Fathom Tunnel (Ward et al., 1976) will be presented here.

In that case, instrumentation displacement results are available for a section without support, as well as for a sprayed concrete supported section.

Results of the first section can be used to determine rock mass properties, which can be input for analyzing the second section to evaluate stresses in the sprayed concrete. Another reason to use Four Fathom Tunnel results is that the rock mass has pronounced time-dependent behaviour.

Figure 8 shows the result of curve fitting the model to measured displacements \( u \) at tunnel crown in the section without sprayed concrete. No large effort for curve fitting was made because the accuracy of instrumentation results available in graphical form is not very good. Excavation rate of 2.8 m/day has been assumed. The sharp transition at about \( u=12 \text{mm} \) is not due to rheological reasons. When the excavation face reaches the distance of 1.72\( R \) to the instrumented section, according to equation 3, no more load is transferred to that section, and the displacement increase rate drops sharply. This is a deficiency of the
equation proposed by Schwartz and Einstein (1980) as a smooth transition is more realistic. Adjusted parameters were $G_1 = 700\text{MPa}$, $G_2 = 230\text{MPa}$, $\eta_3 = 6 \times 10^7\text{MPa} \cdot \text{s}$ and $\eta_2 = 2 \times 10^7\text{MPa} \cdot \text{s}$.

Those values were used for curve fitting the model to instrumentation results at the concreted section (Figure 9). Adjustment has been achieved by varying the final value for the deformation modulus $E_{28}$ that takes into account other strain components. A value for $E_{28}$ of 17GPa was found.

Finally, Figure 10 shows the variations of sprayed concrete stress with time for two different excavation rates (2.8 m/day and 10 m/day).

The strength variation of J3 sprayed concrete as proposed by the Austrian Concrete Society is also presented for illustration. The dependence of the stress on the excavation rate becomes evident. It is also clear that the excavation rate of 10 m/day would be unfeasible for this tunnel.

### 4.2 Three-dimensional finite element analysis

Gomes and Celestino (2009) presented results of parametric 3-dimensional finite element analyses to investigate the load transfer mechanism to the support at tunnel face as a function of the relative stiffness between support (constant properties) and the geometrical parameters shown in Figure 11: the excavation length $L_e$ and the initial support delay length $L_{ui}$. The length of the support element $L_S$ is equal to the excavation length $L_e$; the final delay length $L_{uf}$ is given by:

$$L_{uf} = L_{ui} + L_e$$

(12)
Figure 11 Longitudinal section with geometrical element definitions

It is important to note that three-dimensional analyses allow not only to evaluate internal forces and displacements, but also the excavation stability including zones with early age sprayed concrete with low strength.

Typical results of internal forces are presented in Fig. 12 for a circular tunnel. In that figure, $T^* = \frac{T_{FE}}{T}$ where $T_{FE}$ is the normal force obtained by finite element, therefore considering the delay in the installation of the support and $T$ is the normal force obtained using a closed-form solution in plane strain condition. $Z$ is the distance from the tunnel face measured along the tunnel axis.

It is interesting to note the high value of normal force close to the face, indicating stress concentration. Schwartz and Einstein’s (1980) results do not show that high values, probably because their mesh was not fine enough. Müller (1990) found evidence of that high stress in model tests.
In Figure 12, $L_d/R$ was kept constant; the value for $L_d/R$ varied from zero to 0.5. The influence of $L_{ui}$ on the normal force is quite significant, even though some authors (e.g. Schwartz and Einstein, 1980) consider only the influence of $L_d$ which was kept constant in this case. A summary of variation of the final load transferred to the support (far away from the excavation face) is presented in Figure 13. The dimensionless normal force $T^*$ is presented as a function of the delay of support installation and the coefficient $C^*$ indicating the relative stiffness between ground mass and support given by:

$$C^* = \frac{E_R (1-\nu_i^2)}{E_s A_s} (1-\nu^2)$$

(13)
An evaluation of the influence of the time-dependent behavior of the sprayed concrete was carried on by Gomes (2006) and is summarized here. Only the variation of the elastic modulus with time is considered according to the equation proposed by Golser (1999):

\[ S^*(t) = \frac{1}{22.5} \left[ (1 - \alpha) \cdot \frac{\tau}{25 + 1.2t} + 3\alpha \cdot \frac{\tau}{100 + 0.9t} \right] \]  

(14)

where:

- \( S^* \) is the ratio between the values for the elastic modulus at age \( t \) and at 28 days;
- \( t \) is the age (days);
- \( \alpha \) is the ratio between the current normal stress and current uniaxial compressive strength.

Considering the advance rate as \( V \), the time required \( \Delta t_s \) for the completion (excavation and support installation) of one round is:

\[ \Delta t_s = \frac{1}{V} \]  

(15)

The number of the round \( N_t \) at a distance \( Z \) from the face is:
The time required to complete \( N_L \) rounds is:

\[ t = N_L \Delta t_s \]  

Substituting Equation 11 into Eq. 8 one can obtain the elastic modulus at a distance \( Z \) from the face as a function of \( \Delta t_s \) (i.e., advance rate) and the elastic modulus at 28-day age \( E_{28} \).

Figure 14 shows the variation of elastic modulus as a function of the distance from the face and advance rates for \( L_e = 1.0 \text{m} \).

![Figure 14](image)

**Figure 14** Variation of elastic modulus for different excavation advance rates

In Figure 14 it can be observed that the higher the excavation rate, the younger the sprayed concrete and the lower the values for the elastic modulus.

Gomes (2006) analyzed a number of cases combining different ground mass parameters (elastic modulus \( E \) and Poisson’s ratio \( \nu \)). Tunnel radius \( (R = 3.0 \text{m}) \), support thickness \( (t_s = 0.2 \text{m}) \), support elastic modulus and Poisson’s ratio \( (E_S = 25 \text{ GPa}, \nu_S = 0.2) \) were kept constant. The parameters adopted for the ground mass are presented in Table 1.

<table>
<thead>
<tr>
<th>Case</th>
<th>( E ) (GPa)</th>
<th>( \nu )</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.05</td>
<td>0.15</td>
</tr>
<tr>
<td>2</td>
<td>0.50</td>
<td>0.15</td>
</tr>
<tr>
<td>3</td>
<td>1.0</td>
<td>0.30</td>
</tr>
<tr>
<td>4</td>
<td>10.0</td>
<td>0.30</td>
</tr>
<tr>
<td>5</td>
<td>50.0</td>
<td>0.30</td>
</tr>
</tbody>
</table>
Materials corresponding to the ground parameters adopted range from soft soil to hard rock. The initial state of stress is assumed to be isotropic ($k_0 = 1$).

Two similar cases with both time independent and dependent properties are shown in Figures 15 and 16.

The influence of the excavation advance rate can be evaluated by comparing the first curve of Figure 9 ($L_{ui} = 0$) with the results of Figure 15. Final values of normal forces for advance rate of 6 rounds per day are 25% lower than those for 1 round per day and about 30% lower than those for constant properties.
5 Conclusion

The zone of tunnel support with early-age sprayed concrete is important for both support structure design and excavation stability. Rigorous analysis of all the phenomena involved (thermal, chemical and mechanical) including the influence of the loading history on the mechanical properties is still not practical for regular design. Simplified analyses can be very useful to improve the level of safety assessment. A brief summary was presented about the behavior of early-age sprayed concrete. Even though the literature presents more data about cast concrete, there is already enough information obtained from sprayed concrete. Some differences exist. For example, it can be concluded that coefficients of thermal expansion for sprayed concrete are lower than those for cast concrete. Results of in-situ model tests of sprayed concrete were presented including temperature, load independent and dependent strains. The evolution of the deformation modulus was presented for ages ranging from a few hours to 12 days. The influence of the ability of the ground mass to remove water from the sprayed concrete by suction was shown. When that occurs, large shrinkage takes place and the long term deformation modulus values are affected. Values are about 30% lower than when concrete is sprayed on a relatively impervious surface. Procedures for analyzing the behavior of tunnel sections close to the excavation face, when the sprayed concrete is still young, were presented in axisymmetric and three-dimensional conditions. For the axisymmetric case, the procedure was validated with instrumentation data of a tunnel excavated through time-dependent ground mass. The three-dimensional procedure can be used for the analysis of the excavation stability in addition to support design and displacements evaluation.

References

Barla, G.; Bonini, M.; Debernardi, D 2008, ‘Time-dependent deformations in squeezing tunnels’. 12th Int. Conf. of International Association for Computer Methods and Advances in Geomechanics (IACMAG), Goa


Rock Support: Degradation and Failure

John Hadjigeorgiou, University of Toronto, Canada

Abstract
A successful rock support system is required to keep an excavation operational and cost effective for a specific time period. This requires the design and implementation of a system that should ensure the safety of personnel and equipment for the working life of an excavation in a changing and dynamic environment. In this context, predicting and ensuring the long term performance of rock support is a primary objective and a major challenge.

A rock support system employs a variety of reinforcement and surface support components to maintain the structural integrity and control the level of what is considered acceptable deformation in an excavation. All reinforcement and surface support elements are susceptible to degradation over time, and potentially failure. It is useful to distinguish between different types of failures. The first type of failure refers to a particular element failing while the second type of failure implies noncompliance with certain performance criteria, such as operating life, operating limits and requirements. A component failure, may lead to a redistribution of load and result in failure of other components that may lead to a system failure. It is possible for a multiple component failure not to result in a system failure. An explanation for this is that there is an inherent degree of redundancy in the design of support systems.

A rock support system is considered to have failed if the stability of an excavation is not maintained throughout its working life. Failure of support can be sudden such as in a rockburst or a more gradual process characterized by degradation of any component or of the system, such as due to corrosion, loss of tension, and changes in ground conditions. The definition of failure, or serviceability of support, may differ based on the type of excavation and from operation to operation depending on the corporate culture.

1 Introduction
A successful rock support system is required to keep an underground excavation operational and cost effective, for a specific time period in a changing and dynamic environment. This requires the design and implementation of a system that should ensure the safety of personnel and equipment for the working life of an excavation. In this context, predicting and ensuring the long term performance of rock support is a major objective and challenge.

There is a wide range of reinforcement and surface support elements that can be used to maintain the structural integrity of an excavation in rock. Recently, Li et al (2014) provided a review of conventional and energy absorbing rock bolts. Conventional rock bolts include mechanical bolts (i.e. expansion shell bolts), fully-grouted rebar and frictional bolts (such as split set and expandable bolts). Energy-absorbing rock bolts, often referred to as yielding bolts, such as the cone bolts, D-Bolts and Yield-Lok bolts are favoured for rockburst and squeezing ground conditions. A comprehensive review of surface support elements, including steel mesh, straps and shotcrete has been provided by Hadjigeorgiou and Potvin (2011) while Louchnikov et al (2014) reviewed surface support for deformable conditions.

In a successfully implemented rock support system, individual reinforcement and surface support components work together to maintain the structural integrity and control the level of what is considered acceptable deformation in an excavation. The reinforcement and surface support are part of an integrated system that transfers and shares load between the interacting components.

An important consideration in the selection, design and implementation of rock support is the anticipated service life of the operation. In a mining context, service life can be qualified in the following categories:
• Short-term (less than 1 year), e.g. crosscuts, temporary openings.
• Medium-term (1–3 years), e.g. exploration drifts.
• Long-term (more that 3–10 years), e.g. level accesses, ventilation drifts.
• Life of the mine (more than 10 years), e.g. main accesses, ramps.

The traditional challenge in the design of support has been to quantify the potential stress, gravity and seismic loads that acting on the reinforcement and surface support elements, resulting in complex loading mechanisms. Although there are several engineering tools for the design of support system it would appear that most mines rely on local experience to select and modify over time their ground support strategy. Examples of the evolution of support strategies in deep and high stress mature mines have been described by Counter (2014), Morissette et al (2014), Yao et al (2014), Mercier-Langevin & Turcotte (2007), Jacobsson et al (2014) and in squeezing ground conditions by Potvin & Hadjigeorgiou (2008).

The main approaches for the design of ground support are analytical, empirical, limit equilibrium and numerical modelling. However, none of these approaches take into account the degradation of ground support over time. This paper addresses fundamental issues of degradation and failure of support and the impact of these events on the long term stability of underground excavations in rock. It introduces different strategies that may be of use in anticipating the impact of degradation before it results in failure of ground support. Finally, an example is presented illustrating one way to acknowledge the role of degradation. This discussion has important practical implications in the design of support and in developing appropriate long term and rehabilitation strategies.

2 Capacity of Reinforcement and Support Elements

The capacity of an individual reinforcement or surface support element can be determined based on its material characteristics and the subjected loading conditions. It is expected that all products comply with the appropriate national or international standards, such as ASTM. The performance of rock bolts, and to a lesser degree of surface support, can be determined by laboratory and field tests. The practical limitations of testing of individual elements under static and dynamic conditions have been widely reported in the literature, including by Thompson & Villaescusa (2014) and Hadjigeorgiou & Potvin (2011).

In effect, even the most sophisticated laboratory set-ups rely on axial loading of either reinforcement or surface support. This is not what is observed in the field whereby the multitude of loads (stress, gravity, seismic loading) acting on a ground support system result in complex mechanisms that are difficult to quantify. An understanding of the role of degradation can be provided by a series of in-situ pull tests over time. This can be part of the mine QC/QA program.

3 Degradation and Failure

Any reinforcement or surface support element is susceptible to degradation and potentially failure. From the moment a reinforcement, or support element, achieves its initial performance level it is susceptible to degradation. The degradation process can be due or accelerated due to a multitude of factors including:

• Material quality and the presence of manufacturing flaws.
• Installation issues such as bolt orientation, grout quality etc.
• Corrosion of support systems if exposed to corrosive environments.
• Blast damage associated with explosive gases and flyrock.
• Mine induced seismicity resulting in rockbursts.
• Overload of individual reinforcement or surface support elements.
• Damage to reinforcement and support caused by equipment.
If a component degrades, but does not reach the critical performance level during its service life, it is considered to have experienced loss of performance. For practical purposes, it is not considered to have failed. Once a reinforcement or surface support element degrades to a critical performance level it is considered to have failed. Defining what constitutes a critical performance level can be based on observations, for example loading of the screen, or quantitative such as exceeding a certain level of deformation.

It is useful to distinguish into types of ground support failures. The first type of failure refers to a particular element failing, while the second type of failure implies noncompliance with certain performance criteria, such as operating life, operating limits and any other defined requirements. A component failure, may lead to a redistribution of load and result in failure of other components that may lead to a system failure. However, in a mining context, it is possible for a multiple component failure not to result in a system failure. An explanation for this is that there is an inherent degree of redundancy in the choice and design of support systems.

A convenient and simple way to interpret degradation of rock support is the distinction between consumption of capacity due to increased demand and loss of capacity due to damage of support. In the first case the support is working as intended until a threshold is reached where the demand is greater than the capacity, while in the second category damage to support results in loss of capacity. This is summarized in Table 1.

### Table 1 Causes for degradation of support

<table>
<thead>
<tr>
<th>Increased Demand</th>
<th>Damage to support</th>
</tr>
</thead>
<tbody>
<tr>
<td>Changes in stress</td>
<td>Poor installation</td>
</tr>
<tr>
<td>Rock mass degradation</td>
<td>Corrosion of support</td>
</tr>
<tr>
<td>Increase in excavation dimensions</td>
<td>Blast damage (flyrock)</td>
</tr>
<tr>
<td>Mine induced seismicity</td>
<td>Equipment damage</td>
</tr>
<tr>
<td>Blast loading</td>
<td></td>
</tr>
</tbody>
</table>

### 3.1 Examples of degradation and failure of ground support

A series of examples of degradation and failure of ground support are provided in this section. The degradation of ground support in a deep and high stress mining can be a time dependent process. In effect, the installed support can be both appropriate, and adequate for the original ground conditions. As mining advances, the resulting changes in stress can result in rock mass degradation around the excavations. This is manifested by the initiation, accumulation and growth of stress induced fracturing. Under these conditions the ground support system assumes the increased load and work. Further stress changes, and rock mass degradation, result in further load on the ground until eventually the reduction in support capacity leads to failure of the ground support. The use of seismic monitoring systems can monitor the evolution of mining induced seismicity over time and field observations can trace the rock mass degradation and the loss of capacity of ground support.

Figure 1a illustrates a heading in a deep and high stress mining following blasting of a nearby stope. The side walls are clearly damaged and hence of reduced capacity but the ground support of a combination or grouted rebar bolts and fibrecrete ensured the integrity of the excavation. Figure 1b, is of the same heading 10 months later. As mining progressed the resulting rock mass degradation and rock support loss of capacity resulted in failure of the ground support.
In seismic active mines ground support is subjected to dynamic loads that can result in damage. An example of a seismic event resulting in degradation of support is illustrated in Figure 2. In this particular case the impact load did not result in failure of the support. On the other hand, Figure 3 illustrates a severe seismic event resulting in failure of the ground support system.
Figure 3  Failure of ground support as a result of a seismic event, after Morissette et al (2014)

An example of a successful application of surface support is shown in Figure 4. The load bearing capacity of screen is suitable for containing small blocks that can detach between reinforcement elements. In this case the screen has not failed although it can arguably be claimed that it has degraded due to the load. Simser (2007) makes a strong case that degradation and failure will follow the weakest link in a ground support system.

Figure 4  Screen contained loose material. Although it degraded it did not fail

An example of degradation of the screen leading to eventual failure of the surface support is illustrated in Figure 5. In this case the sharp edge square plates cut through the wire strands of No.7 gauge screen. This damage to the screen resulted failure of the ground support system.
Figure 5  Degradation of mesh wire strands resulting in failure of surface support, after Simser (2007)

The limitations of ground support in squeezing ground conditions have been reported by Potvin and Hadjigeorgiou (2008). Figure 6 illustrates an application of fibre reinforced shotcrete that as a result of deformation has cracked, resulting in degradation of the surface support. Several mines resort to the use of screen over fibre reinforced shotcrete, to contain the large shotcrete plates produced by excessive wall deformation, Figure 7.

Figure 6  Degradation of fibre reinforced shotcrete as a result of high deformations
Several examples of degradation of support requiring rehabilitation in squeezing ground conditions have been reported by Hadjigeorgiou et al (2013). For example in Figure 8 the ground support failed, and the excavation required rehabilitation. However, even at high reported deformations, $\varepsilon$ close to 35%, the excavation remained operational and hence did not fail. Figure 9, is a case study where excessive deformation rendered the drift non-operational.

Figure 7  Screen retaining degraded fibre reinforced shotcrete, after Potvin and Hadjigeorgiou (2008)

Figure 8  Failure of ground support due to squeezing ground required rehabilitation but the excavation remained operational thus did not fail
The role of corrosion in degradation of support has been reported in the technical literature. The challenge lies in estimating the reduction in working life of support and determining whether degradation in support capacity will result in failure of the ground support. Figure 10 is an example of degradation that did not result in failure of the ground support system. On the other hand, Figure 11 is an example where the degradation of support due to corrosion, resulted in a gravity driven failure.

Figure 9  Failure of ground support due to squeezing ground required rendered the excavation non-operational

Figure 10  Corroded support resulting in degradation of support capacity without resulting in failure of ground support
Figure 11  Corroded support resulting in degradation and failure of ground support

Figure 12 is an example of equipment induced damage resulting in degradation of support. In this particular case, the scoop had scraped off both reinforcement and surface support meant to confine the pillar. Despite the excessive damage to the ground support, the pillar did not fail.

Figure 12  Degradation of ground support as a result of equipment damage

An example of fly rock damage on surface support is illustrated in Figure 13. In this particular case the push plates were peeled off and the screen was damaged following a development blast.
3.2 Failure of reinforcement elements

There is value in investigating the failure of ground support that results in falls of ground. This information can provide valuable insights on the performance of different reinforcement elements and can result in changes in the support strategy. Failure of rock bolts can be due to tension, shear or in most cases as a result of a combination of tensile and shear loading.

Figure 14 is of an expandable rock bolt that was loaded, and failed under tension. In this case, the bolt experienced localized surface reduction along the fracture surface. An example of a split set failing in tension is shown in Figure 15. This was from an underground hard rock mine displaying squeezing ground conditions.
Li (2010) provided some excellent examples of reinforcement element failures, under high stress conditions, in a cut and fill mine. This is illustrated in Figure 16, where a rebar bolt was subjected to shear loading resulting in an offset of 40 mm and opening displacement of at least 20 mm.

Another example of stress induced bolt failure is illustrated in Figure 17, whereby the expandable rock bolt failed in shear. At this particular location a 22 mm resin rebar bolt, in the same photo, was shown to bend but was not sheared.
In the majority of situations, rock bolts are more susceptible to fail due to a mixed loading mechanism. Figure 18 illustrates a rebar bolt exposed on the advance face of a cut-and fill mine stope. The bolt was subject to both pull and shear loads prior to failure. The axial opening displacement being about 30 mm and the lateral displacement about 18 mm at the position of failure Li (2010).

An example of severe corrosion leading to practical disintegration of the support is shown in Figure 19. In this case both the strap and the split set failed, providing no reinforcement or support.
Simser (2007) has provided several examples of reinforcement and plate failures in some Canadian mines. Hadjiegeorgiou et al (2002) demonstrated how fracture analysis, using macro photography and microscopic analysis using a scanning electronic microscope and metallography can provide more information on the cause of failure.

4 Conceptual representation of degradation and failure of ground support

The long term performance of ground support is influenced by how a system degrades, as well as whether and when it is anticipated to fail. This can allow an operation to be proactive and undertake the necessary rehabilitation. This, however, is not a trivial process. A conceptual representation of the degradation of individual elements or of a support system is presented in Figure 20. The x axis is the anticipated, or actual exposure time for the support while the y-axis is an indication of the performance of the support system.

The concept of a critical performance level can be defined in several ways. The capacity of an individual reinforcement or surface support element can be determined based on its material characteristics and the subjected loading conditions. Laboratory and in situ tests can provide an estimate of reinforcement capacity. In situ pull testing are part of the QC/QA at most mines and can be used to demonstrate whether the installed ground support meets the design requirements. If the design requirements are met subsequent tests at regular time intervals can indicate if the ground support has degraded over time. Pull tests are performed by mine, contractor or supplier personnel. The frequency of testing depends upon the QA/QC requirements of the mine. Another way to define the critical performance level for a particular case is the degree of tolerance to another metric such as tolerable deformation. What constitutes tolerable deformation can differ significantly from site to site. Hadjigeorgiou et al (2013) provided examples of high deformation that were acceptable at underground hard rock mines.

Figure 20 is an example of degradation in performance of two support systems over time. From the moment a support system achieves its initial performance level it is susceptible to degradation. In the first case, degradation curve a), the system degrades over time until it reaches a critical performance level when it is considered to have failed. This could be for example due to loss of capacity associated with corrosion of support. The second system, b), degrades over time but does not reach the critical performance level during its service life. This system is considered to have experienced loss of performance but for practical purposes, it has not failed. The same trends of rock support system degradation could be in place as a result of stress induced rock mass fracturing, and progressive loss of rock support capacity.
The determination of both the demand and the capacity, particularly under rockbursting conditions is problematic, Stacey (2012). The capacity of a system is difficult to define although, Morissette et al (2014) had some success using passive monitoring techniques of ground support systems exposed to rockburst. The service life of the system is defined by the operational needs of the operation. As illustrated in Figure 21 a major seismic load may result in an impact load and failure of support. In the second example the impact resulted in severe degradation of the support but did not fail, until further degradation.

Figure 20  Rock support system performance over time: a) degradation results to failure; b) degradation results in loss of performance

Figure 21  a) Degradation over time, and due to major impact load resulting in failure; b) Degradation over time, major impact load, and further degradation resulting in failure
Quantifying Degradation and Loss of Performance

Quantifying degradation of reinforcement elements is a major challenge. The most common objective of testing is to ensure that the quality assurance and control (QA/QC) of the products is maintained throughout the working life of the support. A number of tests are routinely undertaken by the mine operators and manufacturers to achieve this objective. In situ testing is undertaken to ensure that the installed ground support meets the design requirements and it has not degraded over time. There are several techniques to test the behaviour of reinforcement and surface support elements in situ. These tests are performed by mine, contractor or supplier personnel. The frequency of testing depends upon the QA/QC requirements of the mine.

5.1 Degradation due to impact load

Large-magnitude seismic events are capable of causing severe damage to excavations and ground support systems. Damage to excavations and/or ground support, triggered by mining-induced seismicity are defined as “rockbursts”. The dynamic capacity of a support system is dependent on the connectivity and compatibility of its reinforcement and surface support elements. Connectivity refers to the capacity of a system to transfer the dynamic load from an element to another, e.g. from the reinforcement to the surface support through plates and terminating arrangements (split set rings, nuts, etc.), or from a reinforcement/holding element to others via the surface support. Compatibility is related to the difference in stiffness amongst support elements. Load transfer may not take place appropriately when there are strong stiffness contrasts within a ground support system. Quantifying the capacity of a ground support system is still a major challenge. Over the years, testing of ground support elements have focused mostly on individual reinforcement and surface support elements rather than on whole support systems.

Although information from multiple dynamic (drop) tests over time cannot be extrapolated to predict the dynamic capacity of ground support systems, it is still of some use for design purposes, Stacey, (2012). Drop tests provide essential information on the behaviour of support elements under tensile dynamic loads and allow for direct comparison between reinforcement and surface support elements, Hadjigeorgiou & Potvin (2011). There is an increasing level of information on the results of impact load of support, e.g. Bucher et al (2013), Player et al (2013) and Doucet and Voyzelle (2012).

An indirect indication of the loss of capacity following an impact load on a reinforcement element can be estimated in laboratory drop tests where multiple loads may be required in order to fail a rock bolt. This is illustrated based on a series of impact loads on a high quality grouted threaded rebar. The tests were undertaken at the CANMET testing rig in Ottawa and followed ASTM D 7401-08. As shown in Figure 22, the first impact load resulted in a split of the tube, but the bolt did not fail. The bolt was subjected to a second impact load and this time failed.

It is fully recognised that this example only demonstrates the degradation-failure process in a reinforcement element, under axial loading in a controlled laboratory environment. It is difficult to demonstrate the same phenomenon in the field, where a ground support system is subjected to more complex loading mechanisms.
Degradation due to corrosion

Corrosion is the deterioration of a substance (metal), or its properties, because of a reaction with its environment. All metals are susceptible to corrosion. From a ground control perspective it is important to identify the rate of corrosion, and how it can affect both short and long term performance of ground support.

In recognition of the importance of potential degradation of support, efforts have been made to identify the corrosiveness of mining environments, for example Hadjigeorgiou et al (2008) and Villaescusa et al (2008). Both atmospheric and aqueous conditions have to be assessed to establish the potential for corrosion of a given area. The influence of iron bacteria and solid mineral deposits on the reinforcement and support steel has been addressed by Hadjigeorgiou et al (2012). A preliminary assessment of the expected levels of corrosion can be used in the choice of reinforcement and support, as well as developing a strategy with respect to expected rehabilitation requirements, Dorion & Hadjigeorgiou (2014).

Quantifying the loss of capacity attributed to corrosion is not a trivial topic. The use if corrosion coupons have been successfully used by Villaescusa et al (2008) in laboratory and field conditions in Australian mines, and in Canadian mines by Dorion and Hadjigeorgiou (2014). The procedures followed ASTM G4-01 and ASTM G1-03 standards. Villaescusa et al (2008), based on measurements in corrosion chambers, provided estimates for minimum and maximum service life of cable bolt strands in high ground water flow environments. Dorion and Hadjigeorgiou (2014) developed estimates for the loss of capacity of friction bolts by extrapolating from work in corrosion chambers, laboratory testing and onsite observations. A series of design charts were also developed to characterize the loss of capacity of #6 mesh under different corrosivity environments. Table 2 provides a link on the loss of capacity of reinforcement and support for reinforcement and surface support elements.

Finally, the loss of capacity of bolts exposed to ‘low to moderate corrosion conditions’ and to ‘moderate to high corrosion conditions’ has been demonstrated by pull tests on Swellex bolts as reported by Charette et al. (2004). A series of design curves were established linking the reduced capacity over time for different corrosive environments.
### Table 2: Linking onsite observations to resulting loss of capacity and required intervention, after Dorion & Hadjigeorgiou (2014)

<table>
<thead>
<tr>
<th>Corrosion Level</th>
<th>Description</th>
<th>Corrosion Rate</th>
<th>Loss of Capacity</th>
<th>#6 Mesh Diam.</th>
<th>Required Intervention</th>
</tr>
</thead>
<tbody>
<tr>
<td>C1: Negligible corrosion</td>
<td>Steel is in excellent condition and corrosion signs only on surface. A few localised spots, less than 10% of the surface is corroded.</td>
<td>&lt;0.02 mm/yr</td>
<td>&lt;10%</td>
<td>&gt;4.75 mm</td>
<td>None</td>
</tr>
<tr>
<td>C2: Localised corrosion</td>
<td>Corrosion is characterised by localised spots on the surface. Between 10 and 75% of the surface is corroded. Steel is in good condition.</td>
<td>0.02 to 0.04 mm/yr</td>
<td>10 to 20%</td>
<td>4.50 to 4.75 mm</td>
<td>None</td>
</tr>
<tr>
<td>C3: Surface corrosion</td>
<td>Corrosion over 75% of the surface. Corrosion is only on surface. If a corrosion crust is present it is very thin. Can identify blisters.</td>
<td>0.04 to 0.15 mm/yr</td>
<td>20 to 35%</td>
<td>4.00 to 4.50 mm</td>
<td>None to follow up.</td>
</tr>
<tr>
<td>C4: Advanced corrosion</td>
<td>100% of the surface is corroded. Can identify blisters. Thin corrosion crust (&lt;1 mm) easily removed.</td>
<td>0.15 to 0.30 mm/yr</td>
<td>35 to 50%</td>
<td>3.50 to 4.50 mm</td>
<td>Follow up. If installed over 12 months it will display signs of severe corrosion.</td>
</tr>
<tr>
<td>C5: Very advanced corrosion</td>
<td>100% of the surface is corroded. Thick corrosion crust (&gt;1 mm) and flaky.</td>
<td>0.30 to 0.60 mm/yr</td>
<td>50 to 75%</td>
<td>2.50 to 3.50 mm</td>
<td>Consider replacement of installed units.</td>
</tr>
<tr>
<td>C6: Extreme corrosion</td>
<td>Corrosion goes through the steel. Integrity of steel has been damaged. Pieces are easily breakable by hand.</td>
<td>&gt;0.50 mm/yr</td>
<td>&gt;75%</td>
<td>&lt;2.50 mm</td>
<td>Reconditioning. May require immediate intervention.</td>
</tr>
</tbody>
</table>

### 6 Implication for Design and Long Term Performance of Support

The implications on the integrity of an excavation of support degradation over time is difficult to quantify. This is a direct effect of the complexity of the loading mechanisms, environmental and ground conditions and the in-situ original ground support capacity. This section provides an example to illustrate a path
forward towards quantifying the loss of capacity due to degradation of the ground support, and its impact on the long term integrity of the excavation.

A case study was prepared to demonstrate the impact of degradation of support on the long term stability of a 5.5 x 5.5 m long drift in the EW direction. This was representative of field data from a mine in the Canadian Shield. In this example, the investigation focused on the degradation of reinforcement over time due to corrosion and not taking into consideration other degrading factors, such as quality of installation, seismic loads, etc.

To illustrate these concepts the following assumptions were made in developing this case study. The rock mass could be captured using Discrete Fracture Networks (DFN), generated from the data in Table 3 and the stability of the excavation was controlled by the structural regime. The rock mass could be adequately represented by the Enhanced Baecher Model, which is typical of rock masses in which joints are planar and joint terminations are observed in the field (Staub et al 2002).

<table>
<thead>
<tr>
<th>Joint Set 1</th>
<th>Joint Set 2</th>
<th>Joint Set 3</th>
</tr>
</thead>
<tbody>
<tr>
<td>Number of joints</td>
<td>38</td>
<td>114</td>
</tr>
<tr>
<td>Mean orientation dip/dip direction</td>
<td>05°/349°</td>
<td>410°/105°</td>
</tr>
<tr>
<td>K</td>
<td>57</td>
<td>35</td>
</tr>
<tr>
<td>Mean trace length (m)</td>
<td>2.01</td>
<td>2.48</td>
</tr>
<tr>
<td>$P_{21}$</td>
<td>1.07</td>
<td>3.38</td>
</tr>
</tbody>
</table>

The Fracture-SG DFN generator (Grenon & Hadjigeorgiou 2015), using the Enhanced Baecher model, was used to calibrate a series of DFN with a typical DFN of 40 x 40 x 40 m having more than 160,000 fractures. The individual stability of every tetrahedral wedge at the face of the excavation was determined based on a limit equilibrium analysis following the methodology described in Grenon & Hadjigeorgiou (2003) and Grenon et al (2016). The resulting wedges at the back of the excavation, for a particular DFN, are illustrated in Figure 23.

Figure 23 Wedges formed at the back for the most critical DFN
In this particular example, only one of the generated rock wedges was greater than 7 tons and only one had an apex length greater than 2.1 m. For the purposes of this simulation exercise, a reinforcement pattern of 1.2 x 1.2 m comprised of expandable friction bolts was employed, Figure 24. The bolts had a 10 ton tensile capacity with a bond of 12 tons/m. The use of pattern reinforcement, using expandable friction rock bolts, resulted in a stable excavation. The installed rock bolts successfully stabilized all large wedges formed in the back of the drift.

Figure 24 Reinforcement pattern for wedges formed at the back for the most critical DFN

The impact of degradation of reinforcement on the long term stability of the excavation was addressed by assuming that the drift was developed in a corrosive environment. Based on previous work on the loss of capacity of support by Dorion and Hadjigeorgiou (2014) three potential scenarios were explored. Referring to Table 3, the corrosion levels would result in a reduction of tensile and bond capacity to 75%, 50% and 25%. It should be emphasised that this is a conceptual example, based on a series of simplified assumptions. The most important assumption is that the reduction in capacity of the installed expandable bolts was due to degradation associated with uniform corrosion.

The largest wedge formed along the drift was 7.8 tons and was defined by the intersection of three joints from joint sets 2 and 3. This wedge resulted in a factor of safety of 0.41 and was therefore unstable. The installation of the expandable bolts in a pattern reinforcement resulted in 2 bolts intercepting the wedge and in a factor of safety of 2.29. In order to explore the impact of corrosion driven degradation of the reinforcement a series of simulations were undertaken. In each scenario, a different reduction in the capacity of the reinforcement over time was used, attributed to different levels of corrosion. The results of these stability analyses are summarised in Table 4. In an extreme corrosive environment a reduction to 25% capacity of the rock bolts would result in a factor of safety lower than 1. For comparison purposes, the factor of safety is also provided for the stability analysis in the absence of any reinforcement.
Table 4  Factor of safety for reduced reinforcement capacity due to uniform corrosion

<table>
<thead>
<tr>
<th>Factor of Safety</th>
<th>100% capacity</th>
<th>75% capacity</th>
<th>50% capacity</th>
<th>25% capacity</th>
<th>No reinforcement</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>2.29</td>
<td>1.72</td>
<td>1.15</td>
<td>0.75</td>
<td>0.41</td>
</tr>
</tbody>
</table>

These series of simulations and analyses, suggest that at a certain level of degradation will result in failure of the reinforcement and instability of the excavation. This analysis is consistent with the conceptual, rock support system performance over time degradation curves, in Figure 20. A practical implication of this analyses is in the development of a rehabilitation strategy. Based on the evolution of the corrosive environment, a mine may develop its own critical performance indicator at which point it would intervene and initiate a rehabilitation program.

A further use of this approach is that it provides an opportunity to explore the impact of different reinforcement scenarios. Consequently, it can further provide an insight into the impact of the use of measures to extend the longevity of the bolts, such as the use of corrosion inhibitor coatings, or other types of bolts.

7 Conclusion

Rock reinforcement and support performance degrade over time. This degradation can potentially result in failure of an individual element or of the ground support system. Several examples of ground support degradation and failure have been provided from underground hard rock mines.

A discussion on the role and impact of degradation of ground support has resulted in a series of conceptual models, accounting for different degradation processes such as stress changes, impact loading and corrosion. These models were further defined based on the concepts of critical performance level and working life of the reinforcement.

Different degradation mechanisms may result in failure or loss of performance. The major challenge lies in quantifying and understanding the impact of degradation and failure on the long term performance of reinforcement. A conceptual case study has been presented illustrating the impact of different degradation scenarios on the long term stability of an excavation. This provides the necessary tools to develop proactive rehabilitation strategies.

Acknowledgement

The author would like to acknowledge the long term collaboration of several mining companies and academic collaborators. Yves Potvin, Brad Simser, Martin Grenon and Philippe Morissette provided valuable comments and input in the preparation of this manuscript.

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Rock support in a cut & fill mine – from the operational perspective of a mine manager

S. Haugen, Boliden, Sweden

Introduction

When I studied mining engineering in the mid-1980s, our professor explained that Cut & Fill mining was an expensive mining method with low productivity and high risks for the operators and not likely to be of any use in the future.

Since then, the Boliden mines have managed to mechanise this mining method and are now achieving safe working conditions and higher productivity. Cut & Fill compares favourably to other mining methods in steep, irregular orebodies in poor ground conditions as long as the ore value is sufficiently high. The successful introduction of a rock support combining systematic bolting with shotcrete, is a major contributor to the survival of Cut & Fill into the modern area of mining.

Rock support is both a facilitator and a challenge in modern Cut & Fill mining.

Rock support in cut & fill mining is expensive

Boliden’s Kristineberg mine produces approximately 670 000 tonnes ore/year with this mining method. Cut & Fill requires a significant amount of development and this year 330 000 tonnes of wasterock will be mined resulting in a total production volume of 1 M tonnes rock. The resulting ore:waste ratio is 2:1.

To achieve this production volume, 120 000 rockbolts are installed and 16 000 m³ of shotcrete applied per year.

The total cost of consumables for rock support is 50 Msek/year. 50 Msek amounts to 14% of the mines total operating cost ex depreciation and add 75 sek to the unit cost per tonne of ore produced. Shotcrete consumables; concrete, accelerator and steel fibers, carries the larger portion of this cost at 40 Msek/year. Bolting consumables; rebar bolts, washers, resin and cement, drill bits and rods, cost 10 Msek/year.

The mine has 17 multi-skilled miners on 4 production crews – a total of 68 miners. On average 55% of the mining crew are engaged in rock support activities; scaling, shotcreting and bolting. The remaining 45% of the miners are engaged in drilling, charging, and loading – activities that provide value for our customer. Out of a total of 170 blue collar workers, 22% are assigned to rock support activities. This equals approximately 17% of the mines total employee costs, including white collar workers.

Table 1 Mobile equipment in the mine

<table>
<thead>
<tr>
<th>Rock Support Equipment</th>
<th>Other Equipment</th>
</tr>
</thead>
<tbody>
<tr>
<td>5 bolting units</td>
<td>3 drill jumbos (drifters)</td>
</tr>
<tr>
<td>2 shotcrete units</td>
<td>1 longhole jumbo</td>
</tr>
<tr>
<td>6 mechanical scaling units</td>
<td>2 charging units</td>
</tr>
<tr>
<td></td>
<td>4 LHD’s</td>
</tr>
<tr>
<td></td>
<td>2 large wheel loaders for loading trucks</td>
</tr>
<tr>
<td></td>
<td>3 smaller wheel loaders for clean up</td>
</tr>
<tr>
<td></td>
<td>1 water truck</td>
</tr>
</tbody>
</table>
The value of this fleet (ex. haulage trucks) at today’s price equals approximately 177 Msek. The value of the rock support machines is approximately 92 Msek, 52% of the total value. Haulage is contracted out and the contractor has access to 8 ore trucks and 2 concrete trucks.

Considering the workload on the maintenance department, 13 out of 28 heavy units are used for rock support and they are all maintenance intensive. Last year (2015) the cost of spare parts for our 2 shotcrete units, 6 scalers and 5 bolters was just above 12 Msek.

**Rock support in cut & fill mining is time consuming**

Our Cut & Fill drifts are mined by drill & blast techniques in a manner similar to drifting. A Mine Operating Center schedules and dispatches each activity in the mining cycle. Table 2 below shows the sequence of operations in our mining cycle together with our standard operating times used for scheduling purposes and the number of equipment units for each operation.

<table>
<thead>
<tr>
<th>Activity</th>
<th>Time/round (hours)</th>
<th># of units</th>
</tr>
</thead>
<tbody>
<tr>
<td>Drilling</td>
<td>4</td>
<td>3</td>
</tr>
<tr>
<td>Charging</td>
<td>2</td>
<td>2</td>
</tr>
<tr>
<td>Blasting</td>
<td>At fixed times</td>
<td>na</td>
</tr>
<tr>
<td>Washing down the muckpile</td>
<td>0,5</td>
<td>1</td>
</tr>
<tr>
<td>Loading (from face to loading bay)</td>
<td>3</td>
<td>4*</td>
</tr>
<tr>
<td>Scaling of roof and walls</td>
<td>2</td>
<td>6 **</td>
</tr>
<tr>
<td>Clean up after scaling</td>
<td>1</td>
<td>3 *</td>
</tr>
<tr>
<td>Shotcreting</td>
<td>1 (+4 curing)</td>
<td>2</td>
</tr>
<tr>
<td>Bolting</td>
<td>8</td>
<td>5</td>
</tr>
<tr>
<td>Scaling of face</td>
<td>2</td>
<td>6 **</td>
</tr>
<tr>
<td>Clean up before drilling</td>
<td>1</td>
<td>3 *</td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td>24,5</td>
<td></td>
</tr>
</tbody>
</table>

*Depending on the amount of loose rock our 4 LHD’s may be used for the first clean up after scaling.

**The two scaling operations share a total of 6 mechanical scalers.
Not all units are utilized at the same time.

As can be seen in Table 2 each round requires 24,5 hours work at the face. Including the one cleanup operation between scaling and shotcreting, but excluding the curing time for the shotcrete, the rock support operations require 57% (15 hours) of the total operation time.

Each round is supported by fiber reinforced shotcrete and bolts before the next round is advanced. All scaling is performed by mechanical scalers. Only the roof and walls are scaled prior to shotcreting and bolting. After the rockbolts are installed the face is scaled. As little time as possible is left between scaling of the face and start of the drilling operation, to prevent more rock to come loose at the face. After shotcreting the heading is left for approximately 4 hours to allow the shotcrete to cure, before boltholes are drilled through the fresh shotcrete. There are 3 fixed blasting times during a day. After blasting the muckpile is washed down by a water truck. LHDs move the muckpile to a loading bay from where the contractor will move the material to the crusher or to a backfill area. After each scaling operation loose rock will have to be removed before the next operation can start. A geologist will visually inspect each
round at some point between blasting and drilling to determine the direction and position the walls of the next round.

**Rock bolting is our bottleneck operation and sets the production capacity of the mine**

As can be seen from the standard operating times and number of equipment units available for each operation presented in table 1, rock bolting is our bottleneck operation. In order to maximize the production capacity, the focus in short term planning and scheduling must be on utilization of the bolters.

All available bolters should be manned at all timed. We must maintain a sufficient buffer of headings available for bolting, so the bolters never will have to wait for a previous operation to finish in the heading. The bolters should also receive priority in the workshop.

We are presently aligning our production planning and control to this reality. Our daily and weekly performance KPI’s have been changed to rounds bolted/shift or week instead of number of rounds blasted. We strive to level out the workload by setting a fixed target for each shift. Our initial target is 3 rounds bolted/10 hour shift resulting in 42 rounds/week. This target will be gradually raised to eventually reach a maximum capacity of 51 rounds/week. Further increase in capacity will require us to shorten the operation time for the bolters or add more units to the fleet. We attempt to have the bolters setting the pace for all other operations in the mining cycle, but we are presently facing some difficulties in slowing down the drilling operation which traditionally has been top priority.

**Rock support in cut & fill mining is short lived**

Figure 1 below illustrates the layout of one of the production areas in the Kristineberg mine. The height of one cut is 5 m, except the bottom drift which is 6 m high. Each round is max 5 m long. Access drifts are driven from the ramp into the ore, and we are able to reach 5 cuts from each access before it becomes too steep for the mobile equipment to negotiate. Horizontal sill pillars are left between stopes in the higher parts of the orebody, commonly when 10 cuts have been mined and the height of the stope reaches 50 m.

![Figure 1](image)

*A longitudinal vertical cross-section through one of the ore bodies at Kristineberg mine showing the vertical subdivision into mining blocks, sillpillars and cuts*
Each drift inside the ore remains open for a limited time before it is backfilled and mining proceeds to the next cut on top of the backfill.

Larger orebodies in the mine are subdivided vertically into production blocks as illustrated in figure 1. As the production capacity in rounds/week at a single face is limited, this subdivision allows the mine to achieve a higher production capacity. The horizontal length of these blocks is between 150 to 175 m. The access from the ramp intersects the ore close to the middle providing two production faces within each mining block. A 150 m long cut is typically mined out in 2-3 months. The mined out drift is backfilled, the new access is constructed and mining moves onto the next cut on top of the backfill.

As the next cut advances all rockbolts and shotcrete that were installed in the roof of the drift below is “mined out” with each round. Bolts and shotcrete in the walls of the below drift survive and continue to provide support. Depending on the turnaround time for backfill and access drifting the required “life” of the installed support in the roof of drifts within the ore is less than 6 months.

In permanent excavations like ramps, shafts and workshops the situation is quite the opposite. Large deformations in the rock mass force us to apply new layers of shotcrete and install new bolts as the existing shotcrete cracks and the old bolts fail. When mining is taking place near those ramps that are close to the orebody we may be required to re-support parts of the ramp several times a year.

The 2.7 m long bolts from the roof of the drifts within the ore end up in the muckpile where they may damage the rubber tires of the mobile mining equipment. The bolts follow the broken ore onto the ore trucks risking damage to ventilation ducts and piping along the haulage drifts to the crusher. Magnets are installed after the crusher to pick up bolts in order to prevent them ripping conveyor belts or blocking the chutes at the skip loading station. Occasionally the odd bolt will get past the magnets causing downtime and costly repairs.

The ore that the mine supplies to the mill comes with steel fiber-reinforced shotcrete from the roof. These fibers are freed in the grinding circuit and the mill needs magnets of their own to catch our fibers before they reach and clog the screens. This is one reason why we do not test non-magnetic plastic fibers in the shotcrete instead of steel.

**Rock support in our cut & fill mine demands the full attention of the whole organization**

Employees at all levels of the organization from managers to operators have a profound respect for our difficult ground conditions and the associated risk of rockfalls. In an average year, 25% of all our reported near-misses are related to rockfalls. This risk attracts a very large part of our attention, sometimes to the point that it overshadows other equally important problems in the operation.

Manager, engineers and supervisors put a considerable amount of effort into monitoring the stability of our workings and designing and adapting the rock support to the actual conditions in the headings.

Boliden’s technology department has dedicated a rock mechanical engineer to each mine. Kristineberg mine has one rock mechanics technician employed fulltime on site. Once every week the rock mechanical engineer and our technician make a tour of our active headings and inspect the rock mechanical conditions. Our four supervisors visit almost all headings during their shift, visually inspection the stability of the heading and the status of the installed support. Once every week the mine captain joins his supervisors for an inspection round. Our surveyors measure roof subsidence while our rock mechanics technician measures convergence of the heading walls.

The weekly and monthly planning meetings revolve around design and support of the headings.

Our miners play a very important role in the decision process concerning the stability of our headings and our rock support. The scaling operator decides when a face is adequately scaled. He or she estimates how much shotcrete we need to order for the subsequent shotcreting operation in the heading. Each miner monitors the stability conditions of their heading during their operation and informs their supervisor of
need for extra scaling, or any other stability related problems. If in doubt they call their supervisor who will join them and discuss the problem to reach a decision.

Three miners, including the chief safety deputy, is dedicated full time to inspection and re-support in ramps and old workings.

Once monthly a committee including the mine captain, mine planning manager, mining engineer, rock mechanics engineer and technician, geologists, surveyor, chief safety deputy, supervisors and experienced miners from each shift meets to review the rock mechanical conditions and decide on the proposed rock support design for each heading.

When we encounter major rock mechanical problems, a smaller committee also involving miners and supervisors will meet, sometimes in the actual heading where the problem is, to decide if and how to move forward.

The ideal rock support
From a mine manager’s point of view the ideal rock support for our Cut & Fill operation should provide:

Safe installation
To preserve the health for our workers the support should contain no toxic or in any ways harmful substances. The equipment must be safe to operate and maintain. The operators should not be exposed to un-supported unstable rock-conditions during installation of the support. The maintenance personal should never have to retrieve a broken-down piece of equipment from under an un-safe roof. Automation and remote operation may be one possible way forward.

Easy and fail-safe installation
The effect of the installed rock support is highly dependent on the quality of the installation. Today’s rock support contains a number of operations critical to the functionality of the support. The operators should be able to assess the quality of their “product” continuously during operation. The equipment should have built in functionality to aid the operator and restrict the possibility of making “human errors”.

Making the decision that the face is “sufficiently scaled” may be a difficult challenge for the less experienced operators of our mechanical scalers.

Shotcrete must be applied in an even layer of the designed thickness. In our operation the shotcrete operator controls the movement of the nozzle manually. We have been searching for a practical way to scan the resulting thickness during the shotcreting operation without success.

Bolts must be positioned accurately and installed at the right angle. This is a challenge when installing long bolts in narrow or low drifts. The holes must be filled correctly with the right mix of cement and the resin rotated exactly right.

The required “service life”
We need long term support in our permanent excavations, like ramps, shafts and workshops, without the need for applying new layers of shotcrete and installing new bolts. Ideally this support should be able to survive the effects of the advancing mining operations. Again this might not solely be a rock support challenge. Positioning our permanent infrastructure at an adequate distance from the orebodies will likely have a large, positive impact.

Inside the ore we need the full effect of rock support for a quite limited time period depending on how fast we are able to advance each cut. Our mining method has some similarities to longwall mining with its advancing hydraulic shields and no permanent support inside the stope. If and how the same principles might be applies to cut &fill mining of near vertical orebodies is a topic for future research.
Fast installation
The required rock support should be installed with minimal delay in the mining cycle. As the rock bolting operation is our bottleneck, any reduction of this operation time will provide an increase in the mine’s production capacity. This could easily justify manning the bolting unit with more than one operator or investing in a more expensive machine. We have yet not found any bolters on the market that fulfill our expectations. i.e. cut our bolting time in half.

For any other operation, including the remaining rock support operations, shorter operating times may allow us to reduce the required number of machines and operators.

Concurrent installation
Every round in our headings is supported before the next round is taken. In poor ground we no longer allow shotcreting or bolting to lag behind the advancing face. If the required support could be installed simultaneously with other operations, our cycle time could be reduced by up to 50% and our production capacity doubled. Providing some kind of temporary mechanical support; shields, at the face while the permanent support is being installed behind the machine at the face might be feasible. This is of cause less difficult if the machine at the face resembles a continuous miner. Thus the solution may not lay in changing the rock support but in converting from drill & blast to continuous, mechanical mining.

In my opinion the perfect rock support is:

Effective immediately
An important aspect of today’s rock support is that both the shotcrete and the cement grouted rock bolts require curing time before providing their designed effect. Resin anchored bolts are used, but these are more expensive and not ideal in heavily fractured rock. Friction bolts have not caught on with us due to their limited shear strength. More rapidly setting shotcrete is a tradeoff between less delay in the heading and the need to transport the shotcrete long distances and get it out of the shotcrete unit and onto the rock before curing takes place.

Easy to inspect and monitor
In order to feel safe at work underground in our mine our employees must be able to trust our rock support practices and installed reinforcement. Visual inspection by our operators and supervisors is an important part of our procedures. Their ability to make the correct decisions is derived from long experience from this mine with its particular conditions. If we were to change our rock support design, let’s say to a less stiff support, these people would have to “re-calibrate” and learn to react to new visual signs.

New technology opens up the possibility to install online extensometers that can be remotely monitored by our expert rock mechanics. A different approach is to install color-coded extensometers, or “tell-tales”, that provides a visual signal to everyone passing by. The visual signal is alternatively a flashing red light.

Cost efficient
Correctly balancing costs, performance and life of our rock support is a perpetual challenge in all mining operations. In civil construction the price is negotiated with a customer that often is a governmentally owned institution. These institutions are financed by public money and may be willing to pay a higher price to complete a politically motivated construction project. Mines need to cover their costs by the income received from selling concentrates or metal on the market to a price that is outside our control.

“Not needed”
Of cause the ideal rock support is the one that we do not need to install.
Large scale mining methods like open stoping, require fewer development drifts that needs supporting, and the stopes themselves are often left either unsupported or backfilled. There is an ongoing search for positions within the orebody that may be mined by large-scale methods.

My own vision for a future mining method for orebodies that today are mined by Cut & Fill involves continuous mechanical excavation and perhaps temporary supports adapted from longwall mines.
Guidance for managing the technical and geological risks for the first tunnel boring machine to successfully complete a long tunnel under the Himalaya’s

M. Knights FREng., CH2M, United Kingdom

Extended abstract

There have been several attempts to use Tunnel Boring Machines (TBMs) to excavate and support road and rail tunnels for transportation, and hydro and water conveyance tunnels in the Himalayan geology. Prior to the Kishanganga HEP these attempts had been unsuccessful with the TBMs becoming stuck in squeezing ground, severe rock movement or adverse geological situations. The additional challenges of climate, remote location and difficult access have also impacted these previous attempts.

The presentation will give an oversight of the 330mw Kishanganga Hydro Electric Project currently in the final stages of construction completion. Also the presentation will focus on the construction of the 23km long Headrace Tunnel using a SELI Double Shield TBM excavated though squeezing ground. i.e. having the potential to entrap the TBM and potentially overstress the precast concrete tunnel segments forming the 5.2m internal diameter Headrace Tunnel. The Headrace Tunnel lies within sediments and volcanics ranging in age from Cambrian to Triassic. One of the geological groups comprises weak to strong meta-sediments with low unconfined compression strength, high horizontal stresses and at depths of 500 to 1400 m i.e the potential for difficult squeezing conditions through which the TBM has to excavate and support the ground.

Stress Hardening Elastic Visco Plastic numerical modelling examined time dependant loading during the initial design stages. The paper describes the risk management/mitigation methodology that was developed by the designers and the client (in conjunction with research carried out by Turin Politechnic) to deal with the geological and construction risks. The design team were required to produce a methodology guidance report that would assist the site team to deal with a range of geological and engineering geological circumstance. This report included the guidance to manage the range of potential risks including those associated with the squeezing ground in the Headrace Tunnel.

Instrumentation was installed in the precast lining as the segments were built. Instrumented tunnel segments were installed every 250 m along the tunnel length. The tunnel was completed by August 2014 with monitoring and interpretation currently ongoing with creep and shrinkage strains currently @ 70-80% complete with rates now tending to zero over time. The conclusion at this time is that the tunnel segments are performing as expected and that time dependant ground loading is largely complete.

The TBM tunnel was built ahead of programme and below budget and has given confidence to Owners and Contractors that TBM methods of construction can be successfully used in Himalayan geology with the same method of excavation and support now currently being used on an adjacent Hydro project in Kashmir.

Impounding of the upstream dam at Gurez should commence later this year.

References

“Design of the Headracetunnel segmental lining for the Kishanganga Hydroelectric Scheme” Ariza, Mullerova, Palmer, Swannel and De Biase RETC Conference Proceedings New Orleans June 2015. Published by UCA/SME Littleton, Denver, Co, USA

“Geotechnical risk management approach for TBM tunnelling in squeezing ground conditions” Swannell, Palmer, G Barla and M Barla. Published Tunnelling and Underground Space Technology Journal 57 (2016) 201-210 Elsevier Ltd
KISHANGANGA HYDROELECTRIC PROJECT - 330 MW, JAMMU & KASHMIR, INDIA

**OWNER:** NHPC LIMITED  
**EPC CONTRACTOR:** HCC & HALCROW CONSORTIUM  
**COST OF THE PROJECT:** $ 800 MILLION USD  
**START OF PROJECT:** 21 JANUARY 2009  
**COMPLETION OF PROJECT:** 13 DECEMBER 2016  
**NAME OF THE RIVER:** KISHANGANGA  
**TYPE OF PROJECT:** RUN OFF THE RIVER

**SALENT FEATURES**
- **GATHERING AREA:** 1,610 km²  
- **PROPOSED MAXIMUM STORAGE:** 8,260 MCM  
- **FULL RESERVOIR LEVEL:** 1,290 m  
- **MINIMUM RESERVOIR LEVEL:** 1,275 MSL  
- **INSTALLED CAPACITY:** 330 MW  
- **GATED HEAD:** 484 m  
- **DESIGN DISCHARGE:** 5,4 MCM  
- **TYPE OF TURBINE:** PEDESTAL  
- **MINIMUM ENSURE FLOW:** 0.5 MCM  
- **COMPRESSOR FLOW:** 9.5 MCM

**PRINCIPAL SCHEME COMPONENTS**
- 370m HIGH, 5000 LONG SPRAWL ON THE LEFT BANK\NORTH OF THE RIVER.
- 50m DEEP CUT OFF WALL BETWEEN DAM AND RIVER GROUND.
- 2,000m T1 SLUICING ON THE LEFT BANK WITH 3 RAMP GATES.
- 20m A/H ACCIDENTAL SPILLWAY WITH TWO VERTICAL GATES.
- 9.2m Dia, 4000m LONG DIVERSION TUNNEL.
- 10.4m Dia CAPACITY 4 MINTERS INDIAN STEEL TUNNEL.
- 22.5 m RESERVOIR TUNNEL WITH 8.75 KM AS DRE AND BREATHE AT 10.06 DIA HORSESHOE AND 1.25 TUNNEL AT 3.2m DEPTH ON SEGMENTAL LINER TUNNEL.
- 13.75m DIAMETER BORE HOLE UP TO 2184 DEPTH WITH 198 DIA DRILLING.
- 4.00 CHALLENGE CLEARLY INDIAN PRESSURE SHAFT LENGTH 1,000 M.
- UNDERGROUND POWERHOUSE OF 3 X 130 MW PEDESTAL UNITS (110x = 268x = 438)
- UNDERGROUND TRANSFORMER CASSETTE HOUSING 350 M. LUMBER SINGLE PHASE TRANSFORMER (75x = 125x = 130)
- 5m OIL, 500m LONG HORSESHOE SHAPE TUNNEL.
- 400 M MIDDLE AND CENTRAL BUILDING.
- 50m OIL 750m LONG BORE HOLE AND VENTILATION TUNNEL.
- NUMEROUS ACCESS HOLES AND CONSTRUCTION DAMMERS.

**CHALLENGES**
- **DAM AREA TEMPERATURE**: REACHED BELOW FREEZING POINT FOR ABOUT 4 MONTHS. TEMPERATURE REPORTED TO -26°C. 56000 RESTRICTED WORKING SECTIONS.
- **POSSIBLE AVALANCHE RISK**: AT DAM AREA.
- **ACCESS TO DAM AREA**: IS CRITICAL AT THE ROAD PASSES THROUGH ENSURING SECTIONS.
- **DESIGN OF THE TIME SECTION OF THE TUNNEL**: IS CHALLENGING.
- **TIGHT CONSTRUCTION SCHEDULE**: OF UNDERGROUND WORKS.
- **SECURITY ISSUES**.

**ACHIEVEMENTS**
- **FIRST TIME**: SUCCESSFUL COMPLETION OF TUNNELING BY TUNNEL BORING MACHINES (TBM) IN INDIAN GLACIATION.
- **RECORD OF 9.5m TUNNELING IN ONE MONTH**.
- **CONSTRUCTION OF DAM AND SPILLWAY IN FREEZING TEMPERATURES.**
- **SUCCESSFUL CONSTRUCTION OF 17.75m DIAM AND 12.5m DEEP SARGE SHAFT IN ARID ENVIRONMENT.**
- **BEST EXECUTED PROJECT FOR CONSERVATIVE THREE YEARS**.

CH2M
Cost increase, causes and improvements for underground roads and railway

P. Lundman, Swedish Transport Administration, Sweden

Abstract

Underground road- and railway projects form a substantial part of the ongoing investments in Swedish infrastructure. A common opinion regarding infrastructure projects, especially tunnelling projects, is that they are associated with large cost increases. Geological issues are often used as an explanation for the sometimes dramatic cost increases. As shown in this paper cost increases are common and extensive in Swedish underground infrastructure projects and the largest occurs during planning. The focus in this paper is to describe cost increase based on the unique feature of underground road- and railway projects. The truly unique parts of underground road and railway projects are identified as the closed room and the geological uncertainties. These unique features contribute to cost increases in the studied projects. However, the vast majority of the cost increase depends on other factors. All projects are, to some extent, associated with uncertainties that can be divided into three groups: Risk, inherent and inflicted uncertainties. Several of the causes for cost increase that have been identified in this thesis may be regarded as example of inflicted uncertainties. From the perspective of the client on road- and railway projects, activities within one project is very similar to those in others projects. Project organisation is temporary and unique, its abilities to improve are generally restricted to the specific project. To achieve a lasting improvement, experience from individual projects must be transferred to a more permanent organisation, such as the parental organisations of the involved actors.

1 Introduction

Modern society needs infrastructure to maintain its basic functions, not only for people commuting to work and school, but for the transport of food and goods to all parts of our society. Infrastructure connects communities and increases accessibility between companies, stores and customers. Investments in the transport infrastructure generally imply that the time for transportation is reduced and that the cost of transportation is lowered. Altogether it means that infrastructure is an important resource that affects productivity and profit of companies as well as nations (Nutek, 2007).

In Sweden, extensive investments in infrastructure for transportation are currently carried out. The investments that will continue for at least the next decade. The majority of road and railway projects are under the Swedish Transport Administration (STA) supervision. STA was founded 2010 mainly by a fusion of the Swedish Road Administration and the Swedish Rail Administration. STA is responsible for the long-term planning of the transport system for road, rail, shipping and aviation and STA is also responsible for building, operating, and maintaining state roads and railways.

Tunnelling projects and underground stations form a substantial part of the total investments in both present and future projects within STA. During the last 20 years the tracks have become straighter and less inclined due to the demand for heavier freight and higher speeds for passenger traffic. One consequence is that the number of tunnels has increased and new tunnels are often longer than older tunnels, see figure 1. The Swedish Transport Administration currently operates approximately 160 tunnels with a total length of approximately 140 km.
During the next decade, new underground infrastructure (tunnels and stations) in the amount of 25 – 40 billion SEK (approximately 2.7 - 4.3 billion EUR) are planned under the auspice of STA. Recently the Hallandsas tunnel have been opened for traffic as well as the Northern Link in Stockholm. Some other projects with a large part underground that currently being built and/or planned are:

- The Citybanan commuter train tunnel under Stockholm city – a 6 km long, double track tunnel constructed close to existing subway tunnels and stations, as well as near other infrastructure tunnels.
- The Bypass Stockholm road tunnel in the western part of Stockholm City – 18 km long, triple lane tunnel to be constructed partly under the lake Malaren.
- The Eastern Link road tunnel in the eastern part of Stockholm – approximately 5 km long immersed concrete tunnel or 6 km rock tunnel.
- The Western Link railroad tunnel under Gothenburg city – a 6 km long, double track tunnel and three underground stations to be constructed close to existing central station and other infrastructure tunnels.
- The new highspeed railway consisting, amongst others, of an underground station close to the Landvetter airport and several tunnels.

These massive investments mean that the demands on STA, both as a developer/builder and as manager/operator of completed tunnels and stations, have substantially increased.

2 Projects

Underground projects for road and railway have much in common with road and railway projects above ground as well as other projects in general. It is essential to characterize a road and railway project, due to
the widespread use of the word “project”, before outlining the unique feature of underground road and railway projects.

2.1 Road and railway projects

It is often stated that projects represent something new, associated with high flexibility and efficiency and separated from the ordinary organization. A range of definitions exist for example PMI (2008) defines a project as a temporary endeavour undertaken to create a unique product, service or result.

The temporary nature of projects implies a defined lifespan of the project under which it is carried out by a temporary and unique project organisation with certain goals. Even though there are unique features in projects a large part consists of routine activities (Nylén, 1999). Road or railway projects are unique in the way that the overall task has never been performed before, although routine activities constitute the bulk of the job. Road and railway projects ends when the temporary organization is phased out, though the facility will be used for traffic for decades. Thus the temporary organization may start a new project, with little or no knowledge of the function of the earlier project.

The primary goal of a project is often referred to as function or quality. Project management has traditionally focused on how the defined demand on function or quality is realized within time schedule and budget. The three project criteria are interdependent (Engwall, 2005), meaning that if any one criteria changes, at least one other factor is likely to change. In practice the final deliverable of any project will be a compromise between time, cost and quality.

Projects are evaluated by comparing the outcome and the goal upon completion. In practice this means that projects that are compared with goals established later in the process have a greater chance of success. It also implies that it is easier to evaluate a project with clearly defined goals compared to projects with vaguely defined goals.

Infrastructure projects for transportation are in general characterized by three main features; they are large, visionary and are providing mutual benefits for many users. They are large both with respect to cost, resources and geography. The visionary aspects are due to the fact that the time from idea to finalization is long, often decadal and the outcome of the projects often has a life span of more than hundred years. Several road and railway projects have, after completion, also the character of being a comprehensive infrastructure system and require therefore long-term planning and management as well as stable financing.

The uniqueness of road or railroad projects can be argued. The purpose of the projects under the supervision of STA is to solve a capacity problem either with a road or a railway. In addition, the degree of freedom is limited due to the fact that the road or railway must be connected to existing facilities, both geographically and technically. Furthermore, the layout is already from the beginning largely determined due to internal regulations and standards within STA. One unique aspect, of course, is that the location of the road and railway implies unique ground conditions.

Depending on which and to what extent different constructions, for example bridges and tunnels, will occur in different corridors the final cost can vary considerably. The uncertainties regarding the final cost are therefore reduced considerably in the end of the investigation phase, when a corridor is chosen and the location is determined.

Altogether, road and railway projects are unique with respect to location, though most of the activities have been carried out in several other projects and the content is already on forehand described.

2.2 Underground road and railway projects

Road and railway tunnels have many features in common with other underground infrastructure projects, such as construction material, a long lifespan and the safety aspects. The purpose of roads and railways is to transport freight and passengers over a long period of time. This implies that those projects are designed
with a high safety marginal and for long lifespan. In addition the tunnel must enable self-rescue in the case of emergencies. STA has for example as ambition that it should be equally safe to travel inside a railway tunnel as outside the same, level crossings excluded.

Road and railway tunnels can be divided into three basic tunnel types:

- In-situ tunnels, constructed by removing the in-situ material without moving the ground above.
- Cut and cover tunnels, normally constructed in shallow trenches and then covered.
- Immersed tunnels sunk into water and situated on, or buried just beneath the sea or riverbed.

A tunnel can be composed of all three types; however, the two first types are most common in Swedish road and railway tunnels, with main emphasis on in-situ tunnels in rock. In addition, road and railway tunnels may be further subdivided due to their function, e.g. traffic tunnels, service tunnels, emergency tunnels and stations.

The drill and blast method is by far the most common method in Swedish road and railway projects. Because most road and railway tunnels also are extensively pre-grouted, an even more suitable name for this type of in-situ tunnels would be drill- blast- and grout tunnel. The ratio between drill and blast and TBM road and railway tunnels in Sweden is approximately 100 to 1.

Infrastructure projects for road and railway above and below ground have much in common, and the majority of the description of projects and infrastructure projects are also valid for underground road and railway projects. However, there are a few, yet important features that characterize an underground project and each of them can contribute to uncertainties. The main factors may be subdivided into two groups that are discussed, namely geology and closed room.

2.2.1 Geology

Geology is most often pointed out as unique for in-situ tunnels penetrating a rock mass. Meaning that the bulk of the construction material and loading conditions is given from the start, it is poorly known and cannot be replaced. The condition for bridges is radically different; there, the engineer can choose the appropriate material as well as dimensioning loads. Moreover, the rock mass is composed of intact rock and discontinuities, e.g. joints and faults. In Sweden, tunnels are often situated below the ground water level. As a consequence the water will find its way into the facility due to the discontinuities. In addition, the knowledge about the in-situ material, which you are forced to use, is limited since it is both costly and time consuming to carry out site investigations.

Obviously geology will contribute to uncertainties regarding the total cost for the tunneling work, because questions are generated as to what to do, and how to do it. In the beginning, when almost no specific site investigation has been carried out, the uncertainties are at their greatest. Obviously, the cost estimate will differ greatly between the alternatives, resulting in a large span of possible cost for the project. As the project commence, more site investigation are carried out and the results from these investigations are reducing the uncertainties. In the phase of the detailed design, the uncertainties are reduced to more a detailed level that will not affect the overall cost as much. The geological uncertainties can affect the cost in several phases and at several different points, see figure 2.
2.2.2 Closed room

Another difference from road and railway projects above ground is that the tunnel generates an almost closed room, henceforward referred to as the closed room. The closed room for road and railway implies special restrictions that affect both the project and the administration.

The restrictions due to the closed room may affect the scope of the project during the planning and construction, due to tunnel safety, ventilation and aerodynamic issues. Tunnel safety may affect the size of the section due to special installations in the tunnel such as sidewalks, banisters, signs and lighting. It might also affect the total length of the tunneling system due to changed length of emergency tunnels. Emissions in the tunnels may call for special ventilation. All this goes back to the fact that smoke and emissions in a closed room will cause more problem than elsewhere on a road or railway.

Another issue related to the closed room is aerodynamics, where the piston effect gives rise to aerodynamic problems that may have to be reduced by, for example, shafts, larger section, reduced speed or sealed trains. Beside the safety and aerodynamic issues, the closed room also implies that many of the installations for the superstructure must be customized with respect to the tunnel section.

The closed room restrictions will contribute to the uncertainties about the scope of the project mainly due to tunnel safety and aerodynamic issues. The serial system during construction implies that the geological uncertainties can affect several activities. A phenomenon that can cause discussions during the construction since the current contract, as a rule, only considers disturbances in one activity. It might very well also contribute to a large variation in the tenders since different contractors probably will consider the uncertainties differently.

3 Cost development in underground projects

The cost development has been studied for Swedish road- and railway projects by use of the case study method (Yin 2006). The perspective of the study is the client which is considered to be the Swedish Transport Administration (STA).

The lifespan of transport infrastructure facilities starts with a societal need and often ends more than 100 years later when the facility is phased out. The process of transport infrastructure facilities can be divided into two parts, figure 3. It is the first part, the process for a transport infrastructure project that has been studied.

---

**Figure 2** Geological uncertainties and the different phases and points that can be affected.

*Geological uncertainties*

*Planning and construction*
- Excavation
- Reinforcement
- Water treatment

*Administration*
- Reinforcement
- Water treatment
The studied projects constitute the lion’s share of recently completed major underground infrastructure projects in Sweden. A short introduction to each project is given in table 1.

### Table 1 Projects under study

<table>
<thead>
<tr>
<th>Project</th>
<th>Type</th>
<th>Length</th>
<th>Location and type</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bothnia Line</td>
<td>Railway</td>
<td>190 km, including 15 single track tunnels. Entire length of the tunnels is 39 km, including service and rescue tunnels.</td>
<td>East coast, tunnels in both urban and rural settings, crystalline rock, D&amp;B.</td>
</tr>
<tr>
<td>City line</td>
<td>Railway</td>
<td>Total 7.5 km, including 1.5 km bridge and 6 km tunnel (single and double track tunnels and two new underground stations).</td>
<td>Stockholm, urban, crystalline rock, D&amp;B.</td>
</tr>
<tr>
<td>City tunnel</td>
<td>Railway</td>
<td>Total 17 km, (8 km double track, 2 x 6 km single track tunnels, 3 km single track, two new stations)</td>
<td>Malmö, urban, sedimentary rock, TBM and road-header.</td>
</tr>
<tr>
<td>Hallandsas</td>
<td>Railway</td>
<td>Total 2 x 8.7 km single track tunnels.</td>
<td>Båstad, rural, crystalline rock (strongly weathered partly), TBM.</td>
</tr>
<tr>
<td>BanaVäg i väst</td>
<td>Road and railway.</td>
<td>Total 75 km four line highway and 75 km double track railway, including 5 double track tunnels, total 5.2 km.</td>
<td>West coast, Gothenburg to Trollhättan, rural, crystalline rock, D&amp;B (studied cost concerns the railway).</td>
</tr>
<tr>
<td>Ådalsbanan</td>
<td>Railway</td>
<td>Total 130 km (100 km improvement, 30 km new line), 8 single track tunnels with a total length of 14 km.</td>
<td>East coast, Sundsvall – Kramfors (Bothnia Line), rural, crystalline rock, D&amp;B.</td>
</tr>
<tr>
<td>South Link</td>
<td>Road</td>
<td>Total 6 km, including 2 x 4.7 km two lane tunnel.</td>
<td>Stockholm, urban, crystalline rock, D&amp;B.</td>
</tr>
<tr>
<td>North Link</td>
<td>Road</td>
<td>Total approx. 5 km, including 2 x 4 km three lane tunnel, exits and entrances. Total tunnel length is 9 km in rock.</td>
<td>Stockholm, urban, crystalline rock, D&amp;B.</td>
</tr>
<tr>
<td>Bypass Stockholm</td>
<td>Road</td>
<td>Total 21 km, including 2 x 17 km 3 lane tunnels.</td>
<td>Stockholm, urban, crystalline rock, D&amp;B.</td>
</tr>
</tbody>
</table>
3.1 Cost increase

The construction cost for the projects varies due to their prerequisites. Some of the studied projects are situated in rural Sweden and others in the center of major towns. As a consequence, the unit price for the projects has a large variation. The unit price calculated as cost per km track or lane varies with a factor of 10. The use of historical unit price for early calculation of new projects contains large uncertainties and it is important to compare unit prices from projects with the same conditions.

One might expect that the cost estimates is more complex for projects constructed in city centers with a larger amount of restrictions compared to rural projects. However, there is no evidence that city center projects have larger cost increase (percentage) than rural projects. All projects experienced a general cost increase. Some projects increase linearly with time other stepwise, some projects experience a dip towards the end. The average cost development in the studied project is illustrated in figure 4.

![Average cost development related to estimated cost at the governmental approval](Lundman, 2011).

The gradient in figure 4 is fairly constant until the time of construction, when the gradient vanishes or even become negative. The result is similar with the result reported by the United States GAO (1997), namely that the majority of the cost increase in major projects occurs previous to construction.

3.2 Bothnia Line example

As shown above, it is evident that cost increase occurs in Swedish underground infrastructure projects. However, it seems to be more difficult to determine causes of cost increase than to establish that they occur (Flyvbjerg et al., 2003). To determine causes to cost increase in the Bothnia Line project required a more detailed study concerning the 13 tunnel projects included in the project. The Bothnia Line is a 190 km-long railway in the north of Sweden. The new railway consist of a single-track railway, with 22 passing places, 140 bridges and approximately 40 km of tunnels in crystalline rock.

To be able to retrieve and analyze information on cost increase and their causes a model was developed, see figure 5. The model is based on three sources; Literature, Investigation of cost development in underground projects and interviews with actors in Swedish underground projects. In focus of the model is the unique feature of underground construction projects namely the geological uncertainties and the closed room. The first detailed cost estimates is from the year 2000 when the total cost was estimated to 12 200 MSEK. In the end the total cost was 17 560 MSEK (both costs recalculated in to 2007 price level). This, reveal a cost increase corresponding to 5 360 MSEK. The breakdown of the cost increase is
summarized in figure 5. Only 8% of the cost increase can be traced back to the unique features of underground projects.

As can be seen in the figure the vast majority of the cost increase is due to other causes than the unique features of underground projects even though they have contributed to a cost increase of at least 430 MSEK. The major part of this cost is attributed to reinforcement and tunnel safety.

To further analyze the causes of the cost variation due to geological uncertainties the registered cost variation has been separated in costs for excavation, reinforcement and water treatment on each of the projects. These results were subjected to a basic statistical analysis, assuming that the differences between final- and contracted costs may be treated as normally distributed. The cost increase for excavation, reinforcement and water treatment is calculated as percentage of the contracted cost, see figure 6.

![Figure 5](image)

**Figure 5** Result from the analysis guided by the initial model (Lundman, 2011)

![Figure 6](image)

**Figure 6** Distribution of the difference between final and contracted unit costs for excavation, reinforcement and grouting for the 13 studied tunnel projects in the Bothnia Line project (Lundman, 2011).
The result show that the cost increase associated with excavation work is less than 40 MSEK during construction. This corresponds to 3 % of the contracted tunneling cost. Since excavation cost is not related to the geological prognosis in the contracts, the low variation that is obtained was expected.

The reinforcement has contributed to cost increase for the tunnels. The total cost increase during construction amounts to approximately 110 MSEK, corresponding to 8 % of the contracted tunneling works. Because the reinforcement in the contract is based on the geological prognosis, and the final reinforcement is based on mapping of the tunnel, it is assumed that the difference in cost is a function of the differences between the prognosis and the mapping.

In the majority of tunnels, water treatment is associated with a cost decrease. On the other hand, grouting and especially drains resulted in considerable cost increases in the two most expensive tunnels. The total cost increase during construction amounts to approximately 180 MSEK (grouting – 5 MSEK and drains + 182 MSEK). This corresponds to 14 % of the contracted tunneling works.

When the cost increase attributed to the closed room is analyzed it can be concluded that tunnel safety creates uncertainties that can cause considerable cost increases since the issue is handled on the local rather than on the national level. Furthermore the design of several safety installations with similar function varies considerably between different projects. A higher degree of standardization will facilitate more accurate cost estimates in earlier phases, and also reduce the cost for maintenance.

The focus in the study regarding cost development was to investigate cost increase based on the unique feature of underground road- and railway projects. These unique features contribute to cost increases, however, the majority (>90%) of the cost increase depends on other factors.

4 More systematic learning

The uncertainties for road and railway projects are less than in many other projects. On the other hand, the unique features of underground road and railway projects lead to increased uncertainties, figure 7.

![Figure 7](image)

**Figure 7** General illustration of uncertainties in an underground road or railway project.

If the uncertainties of underground road and railway projects are classified into terms of risk and unknown and combined with cost development in the studied Swedish projects, figure 8 can act as an illustration of the cost development in the studied projects.
Figure 8  Illustration of cost development and decrease of uncertainties as a function of time in underground road and railway project in Sweden.

The risk may be assigned to probabilities for a particular event to occurring. Several explanations for the phenomenon of cost increase by unknown can be considered, for example:

- New information is only gained in such extent that the objective of respectively phase is fulfilled.
- More detailed information transform unknown into known and risk.

Unknown is by definition difficult to consider in cost estimates, though the unknown can be seen as the sum of inherent uncertainties and inflicted uncertainties. The former is defined by a situation in which there are no historical data relating to the situation, the latter is defined by the actors’ inability to learn from previous projects. This means that inherent uncertainty is something that projects have to live with but inflicted uncertainties can be reduced over time.

The organization is temporary and unique in a project and its possibilities to improve are generally restricted to the individual project. There is a huge potential for improvement by forwarding information and experience from old projects into new ones. In order to improve underground projects for road and railway, in any aspect, learning from one project to another by focusing on the similarities between projects, can reduce the inflicted uncertainties. To achieve a lasting improvement, experience from individual projects must be transferred to a more stable organisation such as, for example, the parental organisations of the involved actors, see figure 9. Learning must not only focus on the construction phase; it must take place between the different phases within a project; as well as learning from one project to another.
The parental organization have several different method to successively reduce the uncertainties, for example, improved guidelines, standards and regulation based on a systematic follow up of the outcome of the project. Last but not least by sharing experience between people involved in respective project.

5 Measure to reduce uncertainties with respect to geological uncertainties

Geological uncertainties have been debated over the last 30 years in Sweden. On the other hand, from the perspectives of cost estimates, they are often treated as nonexistent, and rarely discussed in terms of cost and time. Therefore, project managers have little support to put monetary terms on whether there is a need for site investigations or not. Never the less there is an opportunity to improve management of geological uncertainties in underground projects. Sufficient data for analyzing and measuring geological uncertainties exist but are not used to its full potential.

A good starting point is to perform more complete analyses of the data that already exists in the projects. STA have executed studies regarding the geological uncertainties with respect to the reinforcement in different tunnels (Malmtorp and Lundman, 2010, 2012). The aim was to increase the understanding of uncertainties in classification of rock qualities in engineering prognoses, their causes and how they can be reduced. The uncertainties have been investigated through comparisons of engineering prognoses and tunnel mappings from about 6 000 m tunnel from various projects. The differences have been compared to the quality of the geological investigations and analyzed to assess the effect on time and cost in the construction-phase. The results show that uncertainties in engineering prognoses are common in tunnel projects and that they vary extensively within and between projects. The results also show that uncertainties in engineering prognoses affect time and cost in the construction-phase. From this it is concluded that insufficient management of uncertainties partly explains the difficulties to assess time and cost for tunnel projects. Suggested measures in publication of Malmtorp and Lundman (2012) comprise for example requirements for measurements and follow-up of uncertainties in engineering prognoses as well as a model for pre-assessment of uncertainties in engineering prognoses.

In order to measure the outcome and compare results from one project to another it is important to have some type of common platform starting from. STA have a handbook with detailing design guidelines and it has recently been updated, Trafikverket (2015). The handbook presents the view of STA design of the load-bearing structure of rock tunnels. The objective of this handbook is to present practical engineering instructions for tunnel design, as well as clarify the requirements for design in each stage of the planning-,
construction-, and management/operational process. The design guidelines comprise descriptions on, e.g., (i) characterization and classification of rocks, (ii) determination of mechanical properties of rock as input data to analysis, (iii) empirical design of rock reinforcement, (iv) analytical and (v) numerical approaches for stability-, stress-, and deformation analysis of rock tunnels. With the handbook as a starting point measurement and comparison of the outcome from different underground projects become both easier and more meaningful, and the design work become both safer and more efficient.

6 Conclusions

Cost increase has been discussed under a long period of time and it seems to be more difficult to determine causes of cost increase than to establish that they occur.

Geological uncertainties and the chosen ground support affect both time and cost. However, the majority (>90%) of the cost increase depends on other factors; consequently, it is tempting to say that geological uncertainties sometimes are used as a frequent used explanation.

Lasting and substantial improvements of cost estimates and cost management in major underground road and railway projects, need a more systematic approach. A movement from opinions to hard facts on cost increase and its causes is necessary. Such development relies on at least three cornerstones:

- A will and ability to asses and document the final outcome for the different phases as well as for the administration of the facilities.
- A model focused on improvement to guide the work when the symptom of the problem, in this case cost increase, is connected to a cause that can be remedied and
- A lasting organization separated from the individual projects dedicated to the task of lasting improvement in infrastructure projects.

The solution is to handle the uncertainties and transforming unknown to known.

References

Innovative and controversial support for rockbursting conditions

T.R. Stacey, University of the Witwatersrand, South Africa

Abstract

In rockbursting conditions the surfaces of excavations are subjected to sudden ejections, resulting in dynamic loading of rock support, the magnitude and direction of which are unpredictable. It is now well known that yielding support elements are essential if this dynamic loading is to be contained or limited. By yielding, the support is able to contain the ejected rock without failing, which is necessary since failure of a single element of the support system could lead to complete failure of the whole support system.

The development of yielding support elements has progressed over many years, and numerous types of yielding rockbolts are now commercially available. However, other, complementary elements being used with these bolts tend to be conventional elements, such as fibre reinforced shotcrete and wire mesh, without specifically designed energy absorption or yield capability. In this paper, attention is drawn to yielding elements that have been tested in prototype form and shown to have potential, but are not in common use. Examples of such elements are yielding tendon straps and thin spray-on liners. Other methods and ideas that may assist in ameliorating the consequences of rockbursts will also be presented. An example is the orientation of rockbolts with respect to joint orientations to reduce the susceptibility of the bolts to shear failure. A controversial suggestion for rockburst containment is the use of sacrificial support. This implies that support will be designed to fail, and additional elements will also be installed to “catch” the failed support. Additional suggestions for consideration will be outlined in the paper.

1 Introduction

Rockbursts have been a scourge in mines for many years, causing the loss of many lives and substantial damage to mining infrastructure. The main countries in which such occurrences have been experienced include Australia, Canada, Chile, South Africa and Sweden. Recently, severe rockbursts with tragic consequences have been experienced in tunnels in China. Although appropriate design of mining layouts and sequences can assist in the prevention of rockbursts, good rockburst resistant support is necessary to provide safe conditions and contain damage in cases when rockbursts do occur. In this paper, innovative and controversial rockburst support elements and systems, some of which are not in common use in international mining operations, are described. Attention is drawn to yielding elements that have been tested in prototype form and shown to have potential, or have been used on a limited scale.

One of the passions of my erstwhile colleague Dave Ortlepp was the study of rockbursts (Ortlepp 1997), and, as part of this interest, he designed and tested numerous yielding elements – yielding rockbolts, yielding cables, yielding straps and yielding wire mesh. Ortlepp was substantially responsible for publicising and promoting the use of yielding support in rockbursting environments to improve safety, and his designs have not achieved the recognition that they deserved. These designs will be dealt with in the appropriate sections below.

2 Strategic importance of rock support for rockbursting conditions

Rockbursts are hazardous and damaging events. It could therefore be expected that primary design objectives of mining executives would be to prevent rockbursts, and if this is not possible, to contain and
minimise any damage that might result from their occurrence. The design methodology developed by Bieniawski (1992), and represented in the wheel of design in Figure 1, is very relevant in this regard.

![Engineering wheel of design (Stacey, 2009)](image)

In steps 1 and 2 of this process, design objectives should be specified by mining executives and mine management. In step 4 the mechanisms of behaviour (such as source and potential damage locations, magnitudes of events and ejection velocities, directions of ejection forces), should be identified so that appropriate design methods can be chosen, and design criteria set. In step 8, optimisation takes place and includes considerations of costs. Step 10 is important, since monitoring will determine whether the rock support is performing according to the design objectives.

In the case of rockbursts, step 4 of the process is indeterminate, since neither the demand on, nor the capacity of, support systems can be quantified with any confidence (Stacey, 2013). It is therefore necessary to consider conservative amounts of yielding support to cater for “upper limit” estimates of rockburst occurrence. This has cost implications (step 8 of the design process) which will be dealt with in Section 7 below.

3 Yielding rockbolts

Ortlepp (1968; 1969) was probably the first to develop a yielding rockbolt and test it under dynamic loading conditions. The yield characteristic of this bolt is shown in Figure 2.

To test the effectiveness of the bolts, he installed them in a mining tunnel, and used blasting to subject the support to dynamic loading. This tunnel blast test environment, shown in Figure 3, demonstrated that the yielding bolts provided much better support under the dynamic loading conditions than conventional bolts.
Regrettably, Ortlepp’s innovation was never implemented commercially. It is also regrettable that, almost 50 years later, yielding rockbolts are not installed universally in rockbursting mines. Component cost is often a factor in such a decision, since yielding bolts are somewhat more expensive than conventional bolts.

For many years, wire rope bolts in the form of loops were used as “bolts” in deep South African mines. They had inherent yielding capability since their multi-strand geometry, bituminous core and presence of grease that inhibited corrosion, allowed easy lateral compressibility, facilitating yield. Wire rope lacing, which passed through the exposed loops, retained wire mesh surface support, if that was installed. The wire ropes were later replaced by shepherd’s crook bolts, which were easier to install, but did not have the same yielding capability. Nowadays there are numerous types of innovative, commercially-available yielding
rockbolts, (for example, the Durabar designed by Ortlepp (Ortlepp et al. 2001); the modified cone bolt (Simser et al. 2002); the Garford bolt (Varden et al. 2008); the Yield-lok bolt (Wu et al. 2010); and the D-Bolt (Li 2011). Such bolts have proven yield capability under tensile loading at significant velocities, more than 10m/s in some cases. They will also perform their support function under non-rockbursting conditions, for example in squeezing rock conditions (Potvin & Hadjigeorgiou 2008).

To provide greater support capacity than can be provided by rockbolts, Ortlepp designed a yielding cable (Ortlepp & Erasmus 2005), which has been used for rock support in the rehabilitation of a shaft that was expected to be subjected to rockburst loading (Ortlepp et al. 2008). In laboratory testing and underground pull testing, this cable, shown in Figure 4, exhibited a smooth response under quasi-static loading, and a fluctuating dynamic response, with no failure even at a deformation rate of 11 m/s. It does not appear that this cable has been promoted commercially.

Figure 4  Yielding Duracable (Ortlepp & Erasmus 2005)

3.1 Rockbolts subjected to shear failure

It has been reported, after rockburst events, that yielding rockbolts were observed to have failed in shear. This gave rise, in South African mines, to an argument against the introduction of these “expensive” yielding bolts (the original cone bolts), since they were not perceived to perform any better than ordinary bolts in rockbursts. Poor performance of rockbolts under shearing in squeezing rock conditions has similarly been observed (Potvin & Hadjigeorgiou 2008; Turcotte 2010). A limited amount of direct shear testing of rockbolts has taken place, in particular yield rockbolts. A recent publication on the subject is that by Li et al. (2016). It could be expected that, even though yielding bolts might perform somewhat better than conventional bolts, they will not show significant yield under shear loading. In reality, rockbolts rarely, if ever, fail purely in direct shear – there is usually an element of tension in the failure, and the greater the tensile component, the greater the resistance that the bolt will provide to failure. It is suggested that a solution to the limited capacity of rockbolts in direct shear is the following: in addition to a standard pattern of yielding rockbolts, install additional yielding rockbolts in boreholes drilled at acute angles (<30°) to jointing orientations. This concept is illustrated in Figure 5. If shearing takes place on joint surfaces, these bolts will then not be subjected to significant shear loading, but mainly to tensile loading, a condition for which their yield action is designed.

The use of these extra, inclined rockbolts, will strongly enhance the yield capacity of the support system. The cost of the support system will increase, but this additional cost is likely to be offset by savings due to the containment of rockburst damage, therefore making a contribution to the minimisation of consequential costs.
4 Surface support

Types of surface support commonly used include wire mesh, shotcrete (mesh or fibre reinforced), and combinations of shotcrete and mesh. Such combinations could include shotcrete over mesh, mesh over shotcrete, and additional layers of shotcrete. A comparison of several different surface support systems, tested under dynamic loading conditions, is shown in Figure 6.

Figure 5 Yielding rockbolts inclined across jointing to promote tensile loading, and minimise shear loading of rockbolts

Figure 6 Interpreted results of drop tests on containment support (Potvin et al. 2010) showing results of Kaiser et al. (1996) (K); Ortlepp & Stacey (1997) (O); and Player et al. (2008) (P)
Additional results from recent testing can be found in Bucher et al. (2013), Doucet & Voyzelle (2012) and Thompson & Villaescusa (2014).

4.1 Wire mesh

Wire mesh used in surface support includes weldmesh and chain link mesh. Weldmesh tends to be "brittle", often failing at the welds, and shows limited capacity for energy absorption, as can be seen in Figure 6. To remedy this deficiency, Ortlepp developed a yielding weldmesh by introducing “wrinkles” into the wire strands, as shown in Figure 7.

Figure 7  Ortlepp inspecting yielding Duramesh containing “wrinkles” to enhance yielding capacity

Chain link mesh is much more effective in absorbing energy than conventional weldmesh, as can be seen from the results in Figure 6. A very strong chain link mesh is a particularly suitable component in a rockburst support system, and Geobrugg’s Tecco Mesh is an example of this.

4.2 Sprayed liners

Shotcrete, usually with fibre-reinforcement and/or mesh reinforcement, is widely used as a component of rock support. It is very effective under static conditions, confining the rock and inhibiting relative movement of rock blocks (Stacey 2001; Barrett & McCreath 1995; Malmgren et al. 2005). In a rockburst, however, excavation walls are subjected to ejection, and the surface support is subjected to tensile loading. Owing to its brittleness, limited bond strength and low tensile strength, shotcrete quickly cracks and loses its support action. Further, the brittleness of shotcrete can result in shotcrete itself being a hazard during rockbursting if it is not contained by wire mesh, since ejection of “plates” of shotcrete have been reported. This emphasises the importance of the “mesh over shotcrete” surface support combination. In mechanised operations it is usually necessary that mesh has a protective covering of shotcrete to minimise the possibility of tearing off of mesh by the equipment.

In contrast with the performance of shotcrete, a thin spray-on liner (TSL) performed well when exposed to rockbursting (Carstens & Oosthuizen 2004; Carstens 2005). This is probably due to the substantially higher tensile strength of the TSL compared with shotcrete, and to the much greater extensibility of the TSL. Theoretical analyses of the performance of TSLs indicated the importance of the tensile strength of surface support liners (Stacey & Yu 2004). Recent tests have shown that TSLs can enhance the Brazilian tensile
strength of rock by 20-30%, and, when applied on shotcrete, the increase in the Brazilian tensile strength of the shotcrete is about 40% (Mpunzi et al. 2015). These values will depend on the type of TSL used.

Although TSLs were introduced in the 1990’s (Lacerda 1994), they have yet to be satisfactorily recognised as a component of rock support. The EFNARC Specification and Guidelines on Thin Spray-On Liners for Mining and Tunnelling (www.efnarc.org) contain little information relevant to structural support provided by TSLs. An Appendix to the EFNARC document does indicate that there are industry requirements for high performance TSLs and TSLs to mitigate the effects of rockbursts, and that addressing these requirements will be a future development. It is probable that such developments will only result from empirical observations of performance of TSLs under various static and dynamic conditions. In South African mines, TSLs are being used routinely for the purpose of surface support, and, from a fairly recent survey of the main suppliers, annual usage has been in the region of 7500 tonnes of TSL material. Experience of support performance has been good in both static and rockbursting conditions, and in blast resistance, and the use of TSLs in the deep level environment has been proven to be very cost effective compared with shotcrete (Carstens & Oosthuizen 2004; Carstens & Badenhorst 2008). More observations will be necessary to quantify the performance of TSLs under rockbursting conditions.

TSLs, in combination with wire mesh, are suggested in this paper as a potentially valuable support component in combating rockburst damage.

4.3 Straps

In addition to mesh and shotcrete, wire rope lacing over mesh and straps over mesh have been used. Straps commonly used are zero gauge mesh straps, and tendon straps. These additional elements have been very effective in increasing the yielding capability of support systems. As shown in Figure 6 above, drop weight dynamic testing of surface support has shown that wire rope lacing can enhance the capacity by up to seven times (Stacey & Ortlepp 2004), and it is expected that similar results are achieved using straps. Straps also provide a protective layer between faceplates and mesh, preventing localised failure of mesh due to plate action. It is essential, for a successful rockburst support system, that the connecting nuts do not fail, and plates neither fail nor yield significantly (Van Sint Jan & Palape 2007) (see Section 5).

In the significantly deforming rock conditions in the caving asbestos mines in Zimbabwe, conventional straps such as W-straps were found to be unsuccessful as support elements. Instead, tendon straps were developed and patented in Zimbabwe (Wilson 1991). These could accommodate a considerable amount of deformation, as well as being easily integrated into a support system, including shotcrete. These straps were pinned using friction tubes or Split-Sets, and then permanent retention was provided by grouted rockbolts, which were installed through the tubes: “The use of friction tubes allows some controlled debonding of the rockbolts and, hence, there is a distributed yield in the first 0.5m of the anchor.” (Wilson 1991). The “friction tubes” used initially were locally manufactured wheelbarrow handles, later replaced by Split-Sets. This support system appears to be a substantial forerunner of the hybrid bolt now being used successfully in North America (Potvin & Hadjigeorgiou 2008; Turcotte 2010) and the Gabolt in Australia.

Further development of the Zimbabwean tendon straps subsequently took place in South Africa, resulting in the OSRO straps, illustrated in Figure 8, which are now in common use in South Africa and Australia. These straps have demonstrated excellent performance in squeezing and rockbursting environments; their action is summarised in a product brief (www.dsiminingproducts.com): “The strand is able to slip under load conditions through the cross wires to distribute load over several bolts. ... through the use of cross wires tightly “pig tailed” around heavy gauge longitudinal strands ... lessens the risk of tearing away surface support from bolt collars ...”.

Ortlepp extended the yieldability of such straps by introducing “wrinkles” into the longitudinal strands, as in the yielding weldmesh described above and shown in Figure 7. This concept, to increase the yieldability substantially, and therefore reduce the likelihood of containment support failure in dynamic events, warrants greater consideration. Unfortunately these yielding Durastraps do not appear to be promoted commercially at present.
Connecting elements

Although the yielding properties of the retaining and containing support components are extremely important, these elements on their own will not be effective unless the connecting elements between them perform equally well. These connecting elements are the nuts on bolts, the barrel and wedge systems for cables, the face plates, and any other “special” components associated with specific support types. As indicated above, straps are effective in providing a protective “layer” between the surface support and the retaining faceplate and nut/barrel and wedge.

Nuts have been known to fail (Van Sint Jan & Palape 2007), and Potvin & Hadjigeorgiou (2008) have drawn attention to the loss of bolt heads. This is common in squeezing ground, since the conventionally-used bolts cannot tolerate large deformations in the bolt-head region. Deformations and non-axial loading are often much greater in rockbursting conditions than in squeezing conditions, and a guaranteed solution to this bolt head weakness, particularly for the dynamic environment, is yet to be developed. A parallel concept that could have considerable merit in this regard is the Universal or CV Joints that are common in many motor vehicles. CV joints, located in the hubs of the wheels, operate under dynamic conditions, allowing rotation of the wheels, turning of the wheels (steering), and retention in the axial direction. These actions are exactly the properties required of a bolt head that will be subjected to combinations of axial, torsional and shear loadings, both static and dynamic. A bolt-head system incorporating a simple universal or CV joint concept should be capable of handling all types and directions of loading, and overcoming the problem of loss of bolt heads. It would also be beneficial for bolts inclined to the rock surface, as described in Section 3.

An alternative support concept – sacrificial support

Controversial support for rockbursting conditions was conceived more than 20 years ago, and reconsidered more recently owing to observations of rockburst damage in a mine. In Figure 9 a clean concrete surface can be seen in the floor. This concrete was heaved upwards in a rockburst event, severely loading and damaging the substantial steel arch and surrounding concrete in the roof. The damage is extremely localized, since the adjacent arches do not show any distress. They are each about 1m from the damaged arch, and their bases are surrounded by mud and water, not clean concrete.
It may be concluded that the mud and water provided local protection against heave beneath the two adjacent arches. This conclusion is supported by other observations of behaviour in rockburst events (Stacey & Rojas 2013). A particular example, in Figure 10, shows a tunnel support system consisting of concrete panels, each panel being retained against the rock surface by fully-grouted cables. A rockburst caused ejection of numerous panels, but the rock surfaces behind the failed panels were not displaced, and appeared undamaged.

Figure 9  Upwards heave of floor concrete by a rockburst (Stacey & Rojas 2013)

Figure 10  Concrete panels ejected in a rockburst (Stacey & Rojas 2013)
It is considered that the mechanism involved in these ejections could be a tensile wave trap effect (Hino 1959; Starfield 1967; van Heerden 1969; Miklautsch 2002; Grasedieck 2004). A compressive seismic wave passes through the rock/concrete contact and then reflects at the concrete/air interface. On reflection, the compressive wave is converted into a tensile wave, which then travels back to the concrete/rock contact. Since this contact has no tensile strength, the tensile wave is trapped in the concrete layer and the concrete is ejected, leaving the rock undamaged. If mud and water are present, they will be ejected as a “spray” of water and mud, protecting the rock (and the concrete if it is covered by mud and water) from damage. The absence of rockburst damage where the floor was covered with mud in Figure 9 could be explained by this behaviour.

The remedial support solution for the panel system shown in Figure 11 had to be implemented at short notice to ensure safe, stable working areas. It consisted of steel wire ropes, which wrapped over the panels, the ends of the ropes being grouted into boreholes drilled through the panels, as shown in Figure 11. This remedied support system has been subjected to rockbursts, and has prevented the ejection of panels. The panels represent sacrificial support components, and, most importantly, the rock remains apparently undamaged and stable.

Figure 11 Remedial support system with wire ropes (Stacey & Rojas 2013)

Although not yet recognised by the industry as a rockburst support system, it is suggested that a sacrificial support system should be incorporated as part of other systems until its value has been thoroughly confirmed. It could be particularly applicable as a method of floor support, since active support of floors is not common. A conceptual floor support based on the above observations (Rizwan & Stacey 2015) is illustrated in Figure 12. It consists of reinforced concrete floor panels retained by yielding cables (the cables would best be accommodated in recesses or grooves in the panels). The cable anchors would have yield capability, and the overlapping join between the cables from either side would have a similar yield capability (Ortlepp & Erasmus 2005).
Figure 12 Concrete panel floor support concept (Rizwan & Stacey 2015)

7 Consequences and risk

The consequences of rockbursts require very serious consideration. Not only may accidents resulting from rockbursts have tragic consequences, they may also have major direct and indirect economic consequences. For narrow tabular mining operations, Rwodzi (2011) researched the economic consequences of rock falls, which would be similar to those of rockbursts in that mining environment. These are summarised in Table 1.

Rwodzi (2011) demonstrated for rock falls that, in general, improved support, though at increased cost, created considerable value for a mining operation by reducing consequential costs. By installing dynamically-capable rock support, it is certain that the reduction in direct and indirect costs summarised in Table 1 will far exceed the difference in costs between conventional support and high quality rockburst support. In fact, the difference in material costs between conventional and rockburst support elements will be small – using mine costing, it was shown a number of years ago, when the cost of yielding bolts was about four times that of conventional bolts, that, by increasing the bolt spacing from 1m (conventional bolts) to 1.05m (yielding bolts), the same support cost resulted (Ortlepp & Stacey 1995).

Rockbursts have been experienced in mines using caving mining methods (Araneda & Sougarret 2007) and therefore it is appropriate to consider their consequences. Typical costs associated with damage to and loss of drawpoints in a caving mining method can be estimated (Jakubec 2016). Drawpoint construction costs are currently of the order of US$300k-400k per drawpoint. This will vary, depending on the size of mine and size of mining company; for example, $400k for a big project with a big company, and $300k for small project in a country such as Canada. Loss of production can be estimated by considering typical draw rates of 200 to 300 mm per day for a mature cave, and a typical drawpoint area of 225 m². For an ore density of 2.8 t/m³, production loss would be between 126 and 189t/drawpoint/day. For a copper mining operation with typical copper content of 0.7-1.0 %, that is a daily loss of some 3,000-4,000 lb of copper, which, at US$2/lb corresponds with a revenue loss of some $6000-8000 per damaged drawpoint per day. Closure of a drawpoint may influence the efficiency of draw from adjacent drawpoints and also the uniformity of drawdown. Drawpoint rehabilitation costs are high and, because the drawpoint is out of commission, consolidation of the muckpile can take place, creating additional loading on pillars, compounding the damage and the rehabilitation cost. A quoted cost for total drawpoint rehabilitation ranged from US$30,000/m to $75,000/m. Resource loss in a caving operation will be variable, based on the deposit, but an example was the case of a cave that lost 30 out of 300 drawpoints (lost at approximately 50% column drawn), and the corresponding value of the total potential loss of reserves was about $100M. In addition to these identified costs, there will be many other indirect costs, as indicated in Table 1.

Rockbursts have also been experienced during tunnelling. The consequences of some such events have been described by Zhang et al. (2012a; 2012b) and He et al. (2012). These have included casualties, the destruction of a large TBM and a multi-boom Jumbo. In addition, many of the consequences listed in Table 1 will have resulted, incurring substantial extra costs and delays to the progress of the tunnelling projects.

Hence, in mining operations that may experience rockbursts, there is justification for standardising on the installation of high quality rockburst support, and this is likely to result in the creation of considerable value for the operation. It is considered that the decision on the requirement for rockburst support is a strategic one that has a significant impact on safety and profitability, and that it should be made by senior
management on a risk-consequence basis. In fact, risk could be considered as the design criterion. A simple decision-making approach could be adopted: if no indications of rockbursting have been recorded and no rockbursts have occurred, then conventional support will suffice. If rockbursts have been experienced or are expected, then rockburst support should be implemented.

Table 1  Direct and indirect consequences associated with rockbursts (adapted from Rwodzi et al. 2011)

<table>
<thead>
<tr>
<th>Direct consequences</th>
<th>Indirect consequences</th>
</tr>
</thead>
<tbody>
<tr>
<td>Injuries and fatalities</td>
<td>Excessive individual risk exposure for personnel</td>
</tr>
<tr>
<td></td>
<td>Temporary mine closure imposed by authorities</td>
</tr>
<tr>
<td></td>
<td>Medical and rescue operation costs</td>
</tr>
<tr>
<td></td>
<td>Wages and compensation</td>
</tr>
<tr>
<td></td>
<td>Investigations and inquiries – cost of professional time</td>
</tr>
<tr>
<td></td>
<td>Re-training – cost of re-training new employees</td>
</tr>
<tr>
<td>Damage to equipment and machinery (mobile</td>
<td>Safety levies</td>
</tr>
<tr>
<td>and fixed)</td>
<td>Legal costs</td>
</tr>
<tr>
<td></td>
<td>Insurance premiums – Increase due to accident record</td>
</tr>
<tr>
<td></td>
<td>Industrial action – strikes, worker unrest</td>
</tr>
<tr>
<td></td>
<td>Stakeholder resistance (reputation, share price and cost of capital)</td>
</tr>
<tr>
<td>Damage to excavations (access excavations</td>
<td>Loss of production – production affected by equipment loss</td>
</tr>
<tr>
<td>and stopes)</td>
<td>Cost of re-deployment of machinery and personnel to maintain production</td>
</tr>
<tr>
<td></td>
<td>Replacement costs</td>
</tr>
<tr>
<td></td>
<td>Cost of repairs</td>
</tr>
<tr>
<td>Value destruction</td>
<td>Insurant premiums – (Increase due to claims)</td>
</tr>
<tr>
<td></td>
<td>Stakeholder resistance (reputation, share price and cost of capital)</td>
</tr>
</tbody>
</table>

From the examples above it can be seen that investment in the prevention of rockburst damage will create substantial value for a mining or tunnelling operation by eliminating or reducing direct and indirect costs.

8  Conclusions

Rockbursts represent a major hazard to the safety and economics of mining and tunnelling operations. Direct consequences include injuries and fatalities, and damage to equipment and infrastructure. These
direct consequences are associated with financial loss to the operation. However, major financial losses are usually associated with indirect consequences, such as loss of production and loss of reserves.

Conventional design of rock support for rockbursting conditions is not possible because neither the demand on, nor the capacity of, the support are known. The alternative is to adopt risk as the design criterion. This could be based on a simple decision-making approach: if no indications of rockbursting have been recorded and no rockbursts have occurred, then conventional support will suffice. If rockbursts have been experienced or are expected, then rockburst support should be implemented.

Good rock support components to combat rockbursting conditions are commercially available, but it appears that their usage has been limited by high apparent costs. Using a risk-consequence approach to evaluate the financial benefits of eliminating, or at least reducing, both the direct and, particularly, the indirect consequences of rockbursts, is likely to provide justification for increased support expenditure, including for conservative levels of rockburst support. The result will be increased value to the operation.

Acknowledgement

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Innovative and controversial support for rockbursting conditions

T.R. Stacey


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Some aspects on ground behaviour and rock support challenges in difficult ground, from a Scandinavian perspective

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Abstract

Understanding ground behaviour is an essential part of making good rock support design, especially in difficult ground conditions. This paper discusses some aspects on rock support and ground behaviour from a tunnel project related perspective. The importance of knowledge, experience and collaboration is outlined and exemplified.

1 Introduction

As tunnelling and mining people we always strive to improve, and visionary individuals or groups make us go longer, deeper and wider. Boundaries are stretched for every new project we start. Undertakings we manage successfully today would not have been possible some years back. Equipment and material develop, knowledge and understanding develop, and methods develop to make our operations safer, more cost effective and sustainable. Also, development implies that more complex ground conditions can be managed and more advanced support system can be used to control the ground behaviour.

However, we must also remember that the development is based on our history and our understanding thereof, in combination with a constant cycle of feedback, knowledge sharing and improvement. Some historical events where ground behaviour played an important role, and may be regarded as significant milestones, are:

- Collapses in Swedish mines, e.g. Sala Silvergruva and Falu Koppargruva during the 16th and 17th centuries, were quite common and some of them very severe with high numbers of fatalities. The approach to ground behaviour and support design was largely characterised by “learning by doing”.

- The visionary engineer Isambard Kingdom Brunel built the world’s first under-water tunnel in central London, the Thames Tunnel during the 19th century. This was a difficult project with several major accidents, stops and restarts. However, Brunel managed to complete the tunnel by being “innovative and persistent”, as an example he designed one of the first tunnelling shields to protect the tunnel front, and the tunnel was opened in 1843.

- The Gotthard Bahntunnel was built in the 1870 and 1880’s. This was a major achievement since the tunnel was long (15 km) and partly in difficult ground conditions, especially major water inrushes. The engineers really was “exploring new possibilities”, for example by using dynamite for the first time in such large scale. It is interesting to note that the equipment use is not fundamentally different from the equipment we use today.

- A famous event within Swedish tunnelling is the construction of a small diameter tunnel in Stockholm during the 1960’s. The tunnellers experienced a flowing ground collapse in a fault zone and got trapped in the tunnel. This event shows “the importance of being prepared” when tunnelling in complex ground conditions. The tunnellers were rescued and the event ended happily.
• The Hallandsås Tunnel is famous for encountering various problems. The failures during the 1990’s are in the authors’ opinion related to lack of understanding of the “interaction between ground behaviour and methods”. The rock mass at Hallandsås has exposed tunnellers to almost all ground behaviours possible, ravelling ground, running ground, flowing ground, extreme ground water ingress and block instability.

On the engineering and rock mechanical side the following milestones may be regarded as major breakthroughs towards better understanding of ground behaviour and improved design of support systems:

• Karl Terzhagi used ground behaviour as a starting point when he defined his Rock Load Classification System in 1946. This principle is still valid and today’s design engineers could benefit from using Terzhagi’s approach to start with a phenomenological description of the ground.

• NATM was developed and defined during the 1960’s, by Ladislaus von Rabcewicz, Leopold Müller and Franz Pacher, and builds on the very important interaction between rock and support, where the rock mass itself has a significant load bearing capacity. This principle, and the associated Ground Reaction Curve concept, may still be the most important aspect of tunnel and mining design.

• Evert Hoek, Z T Bieniawski and Nick Barton managed to make engineering and mathematics out of natural science when they launched empirical rock classification systems during the 1970-1980’s. This was a major breakthrough as these systems introduced methods to characterise a complex heterogeneous material and means to make rock support design more transparent.

• Numerical modelling within rock mechanics started during the 1980’s. The development within this area is rapid as computer capacity constantly increases. This makes it possible to do very complex analyses and model advanced geometries and use different failure criterion (e.g. Mohr-Coulomb and Hoek & Brown).

• The wider introduction of the Active Design approach and the Observational method in tunnelling during the 1990-2010’s (two decades after the method was first introduced by Ralph Peck 1969) marks a sign of insight that rock mass complexity requires a design approach based on continuous learning, through predictions, observations and planned actions to adopt to actual ground behaviour.

2 The Scandinavian setting

The Swedish tunnelling tradition has its roots in mining. This is a good thing that we still benefit from. The mining industry generally has a better understanding of ground behaviour since the mining operation has a temporary character and the bearing capacity of the rock mass can be utilised to a larger extent, compared to public underground spaces. There is a close cooperation between the mining industry and the civil engineering industry which makes networking and exchange of knowledge easy. In Scandinavia the equipment and material suppliers also have a close cooperation to the industry and this drives innovation and development.

The Scandinavian Peninsula mainly consists of good stable fresh rock. Consequently our support systems are “light weight” compared to e.g. continental Europe. Going back almost 100 years the traditional support system is based on rock bolts (dowels) and shotcrete. A lot of underground openings in Sweden built before 1970 is actually more or less unsupported or support consists of occasional rock bolts. The main purpose of rock bolts is to support a single potentially unstable block. Difficult ground is usually encountered in discrete weakness zones with limited extension (tens of meters rather than hundreds of metres). Here, the rock mass is more fractured or crushed and weathering has occurred creating various gauge materials. Tunnelling through such zones is traditionally done using spiling with small diameter bars (ϕ25-32 mm) and shotcrete arches. In difficult cases ground freezing has also been used.
The primary aim of the support actions have traditionally been to keep the integrity of the underground opening, i.e. to avoid collapses. However, more and more urban tunnelling and development of the urban infrastructure have put higher demands (and nowadays the primary demand) on reducing the impact on the surroundings. Therefore, settlements has become a highly important aspects of an underground project. Since access to underground space, especially in large cities, is becoming more and more scarce we have to stretch the limit and place tunnels and rock caverns in areas with less favourable geotechnical conditions and/or very close to existing structures. Hence, challenges related to stability and settlement increase. This also calls for more robust support measures and during the last decades methods used in continental Europe, e.g. pipe umbrella spiling, jet grouting and permanent concrete lining systems, have been implemented in Scandinavia.

Scandinavian cities like Stockholm, Gothenburg and Oslo are all characterised by a glacial geology where glacial deposits, gravel, sand, silt and clay, fill up depressions in the rock mass. Hence, settlements could be either direct, as a result of ground loss when tunnelling through these depressions in soil or with very low rock cover, or indirect when groundwater leaks into the underground opening and drains the soil layers, especially the clay, on top. This has made sealing of the rock mass through pre-excavation grouting a very important “support” measure in Scandinavia. This has also driven development of grouting methods and material and a lot of research have been conducted in order to improve the understanding of the mechanisms behind rock grouting (Stille 2015). We are today fulfilling demands of leakages down to 1-3 litre/min, 100 m of tunnel. Still, control of water is one of the biggest discussion points in our large projects but nowadays mainly from an operational and maintenance point of view. Dripping water, and as a consequence build-up of ice and icicles, in public tunnels is extremely costly and has triggered a debate in Scandinavia on the Life Cycle Cost of various technical solutions. It is being considered whether or not a higher investment cost, for example by installing water tight lining, pays off in the long run due to much lower operational and maintenance costs.

3 Decision process (related to rock support) in major tunnelling projects

Engineering science aims at doing things. This means that decisions on what to do have to be made continuously and generally it is impossible to predict with absolute certainty what the result of each decision will be. Looking at the construction industry in general and underground construction specifically, it soon becomes obvious that many decisions are very complex. As a consequence a contractor or client may involuntarily take risks that he really wished to avoid. On the other hand, overcautious action may lead to waste of resources in the form of too conservative designs or to a tenderer losing a contract due to too high risk estimates. Thus, it is important to balance the risk-taking and evaluate the consequences of different decisions.

In a geological setting with mainly good and stable rock mass and occasional weakness zones, a key question is “when to be cautious”? The normal setup in Swedish tunnelling contracts is based on a measureable BoQ type of contract. The design is performed by the Client, through his consultant (Engineer), who is also responsible for geological mapping and decisions on final support. The Contractor is responsible for keeping his agreed capacities and building the right quality and normally also for installing temporary support at the tunnel front. This works well in normal and favourable conditions. However, in difficult ground conditions, method and design is more interlinked and there is no clear distinction between temporary and permanent rock support. Furthermore, the initially installed support should interact with the support being installed later. The excavation method also becomes a vital part of the design since the ground interacts with the support system. In these cases a closer collaboration between the project parties is required to get an optimised and safe design and execution. In Scandinavia there are not always such provisions in the contracts/projects which may lead to wrong or not optimised solutions. There is a tendency that the normal project governance, with limited exchange of information between the Client, Consultant on one
hand and the Contractor on the other hand especially in the early phases of a project, lead to too cautious solutions. Or the opposite, where the design and method is not fully aligned leading to unwanted events during execution. There is a need for further collaboration in the industry and a first step would be to allow design engineers to take a more active part in tunnelling during construction to gain experience in practical methods and not least ground behaviour.

The building code in Sweden requires an independent check of complex geo-constructions, which is called GK3 (Geotechnical Category 3). Implementing a good GK3 review system is an excellent way of having the design and construction checked but also promoting discussion and planning of difficult passages or areas. Well managed, this also leads to a good collaboration between the various project parties. In difficult ground conditions a design based on stage-wise progress and a system of tollgates that needs to be formally passed, preferably in combination with the GK3 review, has been successfully implemented in many Swedish projects. This type of “dualistic quality system”, which aims both at doing the right things and doing the things right, was described by Stille & Stille (1997). Before passing a tollgate it must be shown that the observed behaviour of the ground and the support system lies within the predicted behaviour. This way of working is also in line with Active Design and the Observational Method where observations are used to predict the final performance of the design and thereby, in case of deviations, adopt suitable actions before exceeding the safety limit.

Another important aspect is to promote a decision driven information management. We collect and distribute far too much irrelevant information and there is often a tendency within the underground industry to investigate for the sake of investigation. When making geological prognoses and evaluating the value of getting further information the actual decisions should always be the prime concern. One should always ask questions like: - How should the engineering geological information and prognosis be outlined in order to provide a good basis for the specific decisions? - Will the decision change given further information? Decisions on further pre-investigations should always be put in the perspective of how the additional information will be used and what actions will be taken given different scenarios. By making this analysis prior to an investment in an expensive pre-investigation, it is envisaged that money can be saved in a lot of situations. Recent research also presents methods to make stringent analysis of the value of additional information, see e.g. Back (2006) and Zetterlund (2014).

4 Ground behaviour and geological logics

Ground conditions and not least the understanding of ground conditions is vital in order to properly design and execute tunnels and mines. Geological knowledge is the basis for geological modelling and predictions. By possessing a fundamental understanding of geological mechanisms and processes an engineering geologist may draw conclusions even from a limited amount of information. Even though we may consider geological knowledge and geological experience as single methods of describing geological conditions, comparable to other investigation methods, they are more fundamental in nature and required in all activities related to complex underground construction, including for example interpretation of geo-data and assessments on ground behaviour.

Sturk (1998) discussed the concept of geological logics. This means that the complete geological system must be taken into consideration when analyses and interpretations of geological conditions and events are made. There is a logical chain connecting fundamental geological conditions with possible ground behaviour. More specifically we may also say that due to the geological logics certain hazards cannot exist in certain geological environments. A very simplified example of this is that swelling ground may not occur in a homogeneous medium grained sandstone, as it does not contain geological components such as swelling clay or anhydrite. In reality the logical chains are very complex and in order to understand and analyse them geological knowledge is required.

A good way of introducing geological logics into rock support design is to work with geological regimes or domains (Sturk 1998 and Kvartsberg 2013), see Figure 1. Such domain division could aid the design planning process, e.g. by identifying favourable and unfavourable conditions, and the design could be individually
optimised and based on a set of information relevant for the actual conditions. Such set of information gives a broader base than support classes based on a single rock classification system, where specific and relevant geological conditions or features could be missed out. This principle was used during design and excavation of one of the major weakness zones with very difficult ground at the Hallandsås Tunnel Project. In this case four states/classes were defined based on “the Southern Marginal Zone geological regime. These states were defined as including both some important geological parameters and parameters related to the specific construction method in order to create a broad information content, see Table 1.

<table>
<thead>
<tr>
<th>Geological regime</th>
<th>Geological feature</th>
<th>Geological hazard</th>
</tr>
</thead>
<tbody>
<tr>
<td>a) sedimentary rock, sub-horizontal layers</td>
<td>layers of anhydrite (gypsum) unconsolidated layers highly weathered layers highly permeable layers limestone layers with karsts large rock cover, weak rock layers containing gas clayey tuffs</td>
<td>swelling ground running or flowing ground ravelling ground water water squeezing ground gas ravelling or flowing ground</td>
</tr>
<tr>
<td>b) sedimentary rock, sub-vertical layers</td>
<td>soft layers adjacent to hard layers unconsolidated layers highly weathered layers highly permeable layers</td>
<td>spalling or squeezing rock flowing ground ravelling ground water</td>
</tr>
<tr>
<td>c) igneous rock with weakness zones</td>
<td>poor rock in zones smectites in weakness zones permeable rock and crushed zones fractured rock brittle rock and high stresses</td>
<td>running or flowing ground swelling ground water wedges spalling rock</td>
</tr>
<tr>
<td>d) sub-horizontal border, Weak...soil cover</td>
<td>fractured rock and weathered joints depressions with poor rock or soil weathering zones roots crushed zones smectites in zones</td>
<td>wedges running or flowing ground ravelling ground water swelling ground</td>
</tr>
<tr>
<td>e) metamorphic rock</td>
<td>highly deformed bands, poor rock tectonized rock and high stresses fractured rock permeable rock</td>
<td>running or flowing ground squeezing ground wedges water</td>
</tr>
</tbody>
</table>

Figure 1 Examples of geological regimes and domains, from Sturk 1998 and Kvartsberg 2013
Table 1  Excavation states with relevant engineering geological information for the Hallandsås project, Southern Marginal Zone. From Sturk 1998

<table>
<thead>
<tr>
<th>Excavation state</th>
<th>Explicit construction related information included</th>
<th>Explicit geological information included</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>State 1</strong></td>
<td>No tunnelling problems. Full face and full round excavation. Limited rock support.</td>
<td>Fair-Good tunnelling conditions Q &gt; 0.1 Weathering = W1-W2*</td>
</tr>
<tr>
<td><strong>State 2</strong></td>
<td>Limited tunnelling problems. Gallery &amp; bench excavation. Spiling in roof. Systematic rock support (blots and fibre reinforced shotcrete)</td>
<td>Poor tunnelling conditions Q &lt; 0.1 Weathering = W3-W4</td>
</tr>
<tr>
<td><strong>State 3</strong></td>
<td>Tunnelling problems. Very careful excavation by multiple drift and short rounds. Extensive spiling in roof and walls. Extensive rock support (bolts and mesh reinforced shotcrete).</td>
<td>Very poor tunnelling conditions Soil-like conditions Ravelling or swelling ground 0.01 &lt; Q &lt; 0.1 Weathering = W4-W5</td>
</tr>
<tr>
<td><strong>State 4</strong></td>
<td>Severe tunnelling problems. Very careful excavation by multiple drift and short rounds. Extensive spiling in roof and walls. Extensive rock support (steel arches and mesh reinforced shotcrete).</td>
<td>Extremely poor tunnelling conditions Unconsolidated ground Running or flowing ground Q &lt; 0.01 Weathering = W5</td>
</tr>
</tbody>
</table>

State 1 and 2 do not imply any severe tunnelling problems, whereas State 3 and 4 are very difficult to handle and requires careful excavation and extensive rock support. Consequently, State 3 and 4 are time-consuming and thereby also expensive. These four states were deemed to provide a sufficient base for the necessary decisions. Each of them were also deemed to contain site and stage specific information which had a broad acceptance and could be understood by all parties involved.

5 Conclusions and final remarks

Tunnelling, and especially design and execution of rock support, in difficult ground conditions requires special knowledge in order to be successful. Increased challenges in urban areas due to underground space congestion and more general constraints on major projects puts even higher demands on the industry.

The tunnelling industry and its customers want safe tunnels, both during construction and operation. But they also want optimised tunnels from a life cycle cost perspective. The balance of achieving these two major goals is an interesting challenge and requires the industry to develop and improve. This paper have tried to outline some areas in which improvement may have a good effect on the result, in relation to tunnelling in complex and difficult ground and the understanding of ground behaviour.

It is obvious that difficult tasks cannot be solved without knowledge and experience. The continued success for the underground business, not only in Scandinavia, is dependent on attracting young people interested in building knowledge and experience. This is a big challenge in general and the industry must join forces to secure a good regrowth. Furthermore, focus must be put on finding a better balance between theoretical
and practical knowledge. There is a tendency that design engineers lack practical experience, especially related to ground behaviour. This has obvious drawbacks since designs become either too conservative or unsuited related to the actual conditions and the methods used. This can only be solved by further collaboration and closer ties between the parties during execution of projects.

A vital key to better projects and a better industry is, in fact, increased collaboration. The complexity of underground projects necessitates clearly defined responsibilities, structured flow of information, stringency in decision-making and analysis of hazards/risks. All these aspects is influenced and improved by increased collaboration. Role model projects, such as CTRL (High Speed 1) in London, Citybanan and Stigbergsgaraget in Stockholm, give proof that this is the proper way forward. In Sweden there are some very good signs within the construction industry pointing in a more collaborative way of working, e.g. increased number of ECI (Early Contractor Involvement) contracts.

To conclude the keywords used to describe the historical achievements in the introduction to this paper are still valid. Our success and improvement is enforced by “learning by doing” (in a controlled way one might need to add), by being “innovative and persistent” and by constantly “exploring new possibilities”. Further optimised designs will rely on us understanding the “interaction between ground behaviour and methods” and “the importance of being prepared”.

References

Tunneling challenges in young volcanic rocks in Iceland

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Extended Abstract

Iceland is a volcanic island overlying a hotspot on the Mid-Atlantic ridge. The ridge marks the boundary between the Eurasian and the North American crustal plates. The plates drift apart at the speed of 1-2 cm/year and the void between is constantly filled with intrusive or extrusive igneous rock. The active zone of rifting and volcanism crosses the country from the SW corner to the NE. The youngest rock is found near or at the active volcanic zone, but the oldest rocks exposed occur in the easternmost and the northwest parts of the island and are about 14-16 million years old.

Icelandic rocks are generally divided into four main groups: Tertiary plateau basalt formation, Pleistocene basalt formation, Pleistocene hyaloclastite or „móberg“ formation and Holocene formation, which include postglacial lavas and sediments. Basalt is the predominant rock type, with the rock mass largely built up of numerous lava flows, 5-10% are acidic and intermediate igneous rocks and 5-10% sedimentary interbeds.

The tertiary bedrock is characterized by its thin layered nature of the basalt succession, made up of numerous extensive but relatively thin (<10 m) basalt lavas lying on top of each other, often with thin sedimentary interbeds. The single basalt lava typically consists of top and bottom scoria and a dense crystalline central part. The scoria is composed of basalt fragments, partly glassy and partly crystalline, forming a porous breccia. The scoria is usually well consolidated within the tertiary bedrock. The sedimentary rocks of the tertiary basals are mostly composed of thin, finegrained tuffaceous interbeds and occasional thicker conglomerates. The tertiary strata generally dips gently down towards the active zone. Sedimentary rocks are more abundant in the Pleistocene bedrock (10-30%), consisting mostly of sandstones and conglomerates often of glacial origin. The Pleistocene móberg formation is the product of sub-glacial volcanism and consists of various lithological units, ranging from finegrained stratified tuffs through glassy fragmented breccias to pillow lavas and irregularly jointed basalts.

Underground structures in Iceland consist of road tunnels and tunnels/caverns for hydroelectric power stations. 10 road tunnels have been excavated with a total length of approx. 43 km, tunnel length varying from 30 m to 7,9 km. Three road tunnels are currently under construction. 11 hydroelectric power stations utilize underground structures with a total tunnel length of approx. 88 km, tunnel lengths varying from 0,9-70 km. One underground hydroelectric power station is currently under construction. The road tunnels are in the tertiary bedrock while the HEP tunnels/caverns are inland in younger formations.

Excavation and support methods in Iceland have mainly been derived from conventions and experience in the Norwegian tunnelling industry. The Norwegian standard of tunnelling has been the foundation of Icelandic tunnel design, though adapted to local conditions. Conventional drill & blast method is the most common method of tunnelling. TBM has been used in one project so far, a 48 km long headrace tunnel. Tunnels are designed as “single shell” with final rock support typically consisting of B + S(fr) (rock bolting and fibre reinforced shotcrete). The final rock support is selected during construction based on tunnel logging. In very/extremely poor ground conditions, reinforced ribs of shotcrete (RRS) have been used. In recent projects, lattice girders have replaced the RRS, especially in thick layers of weak sedimentary rocks.

Tunnelling in Icelandic rock is met with several challenges such as:

- Variable and rapidly changing ground conditions due to the thin-layered nature of the gently dipping tertiary basalts, frequently resulting in mixed face excavation.
- Weak rock. Thick layers of weak sedimentary rock have occurred in recent projects requiring heavy rock support, major faults (sub-vertical) with weak breccia have collapsed and caused serious delays.
• Water leakages. Tertiary basalts have low permeability in general. The contact between lavas, dike intrusions, faults and joints are water bearing. Unexpected in-burst of water have occurred, in spite of probing. The water is canalized and can be hard to detect.

• Geothermal heat. Some projects have encountered geothermal heat (low temperature) with highest measured water temperature of 65°C. Effects of geothermal heat vary greatly depending on water leakages. When accompanied with water leakages it can have very serious/detrimental effects on working conditions because of high temperature and humidity.
Tunneling challenges in young volcanic rocks in Iceland

Stefán Geir Árnason
### Underground structures

#### Road tunnels

<table>
<thead>
<tr>
<th>No.</th>
<th>Name</th>
<th>Length (km)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Arnardalshamar (1948)</td>
<td>0.03</td>
</tr>
<tr>
<td>2</td>
<td>Strákagöng (1965-1967)</td>
<td>0.8</td>
</tr>
<tr>
<td>3</td>
<td>Oddskarðsgöng (1972-1977)</td>
<td>0.65</td>
</tr>
<tr>
<td>4</td>
<td>Múlagöng (1988-1990)</td>
<td>3.4</td>
</tr>
<tr>
<td>5</td>
<td>Breiðdals- og Botnsheiði (1991-1996)</td>
<td>9.1</td>
</tr>
<tr>
<td>6</td>
<td>Hvalfjarðargöng (1996-1998)</td>
<td>5.8</td>
</tr>
<tr>
<td>7</td>
<td>Fáskrúðsfjarðargöng (2003-2005)</td>
<td>5.9</td>
</tr>
<tr>
<td>8</td>
<td>Almannaskarðsgöng (2004-2005)</td>
<td>1.3</td>
</tr>
<tr>
<td>9</td>
<td>Héðinsfjarðargöng (2006-2010)</td>
<td>11.0</td>
</tr>
<tr>
<td>10</td>
<td>Bolungarvíkurgöng (2008-2010)</td>
<td>5.3</td>
</tr>
<tr>
<td><strong>Total (2010)</strong></td>
<td><strong>43.3 km</strong></td>
<td></td>
</tr>
</tbody>
</table>

#### Hydro tunnels

<table>
<thead>
<tr>
<th>No.</th>
<th>Name</th>
<th>Length (km)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Írafoss (1951-1953)</td>
<td>0.9</td>
</tr>
<tr>
<td>2</td>
<td>Grímsárvirkjun (1956-1957)</td>
<td>0.14</td>
</tr>
<tr>
<td>3</td>
<td>Steingrímsstöð (1958-1959)</td>
<td>0.35</td>
</tr>
<tr>
<td>4</td>
<td>Búrfell (1966-1969)</td>
<td>1.8</td>
</tr>
<tr>
<td>5</td>
<td>Laxá (1970-1972)</td>
<td>1.2</td>
</tr>
<tr>
<td>6</td>
<td>Blanda (1984-1988)</td>
<td>3.4</td>
</tr>
<tr>
<td>7</td>
<td>Sultartangi (1998-1999)</td>
<td>3.8</td>
</tr>
<tr>
<td>8</td>
<td>Kárahnjúkar (2003-2006)</td>
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</tr>
<tr>
<td>9</td>
<td>Kárahnjúkar TBM (2003-2006)</td>
<td>~48</td>
</tr>
<tr>
<td>10</td>
<td>Hraunaveita (2007-2008)</td>
<td>3.7</td>
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<td>11</td>
<td>Búðarháls (2011-2013)</td>
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<tr>
<td><strong>Total (2013)</strong></td>
<td><strong>87.7 km</strong></td>
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</tbody>
</table>

**Icelandic Tunneling Society**
Excavation & support methods

- Conventional drill and blast method.
- NMT in principal
- Final support is selected during construction
  - Fiber reinforced shotcrete
  - Rock bolts
  - Wire mesh / reinforced ribs of shotcrete / Lattice girders / spiles in poor conditions.
- TBM used in one project so far
Tunneling challenges

- Variable ground conditions
  - Mixed face excavation
  - Hard rock, basalt, and weaker rocks

- Weak rock
  - Thick layers of sedimentary rocks
  - Fault zones with breccia
  - Altered rock

- Water leakages
- Hot conditions
Variable ground conditions

Typical sequence of rock units in Tertiary basaltic successions in the Eyjafjörður area northern Iceland

General technical properties of the rock

- **Typical Uniaxial Compressive Strength (UCS) [MPa]**
  - Mainly based on Point load tests
  - Typical Q values
    - Based on tests from the Vatnaleið tunnel project
    - **Main results for reddish sandstone**
      - 20-60 MPa  Q = 0.2-4
      - In upper part of scoria 50-100 MPa  Q = 7-15
      - In lower part of scoria 80-200 MPa  Q = 10-20
    - **Main results for crystalline basalt**
      - 50-250 MPa  Q = 5-20
      - Most frequent 100-150 MPa  Q = 5-15

- **Description of rock**
  - **Crystalline basalt**
    - Bottom scoria often 0.3-1 m
    - Sedimentary interval, normally basic tuff
    - Typical thickness 0.1-1 m
    - Scoria: In upper part, often containing sedimentary fillings, inflated from overlying sedimentary layer
    - Irregular top of crystalline basalt
    - Crystalline basalt, often vesicular in upper part and frequently filled with secondary minerals in the lower Eyjafjörður strata.
    - Crystalline part of a basaltic layer, 70-80% of a tholeiite lava but it is generally over 60% in olivine tholeiite and porphyry lavas.
    - Typical thickness of crystalline part 4-12 m
  - Often containing olivine jointing (diameter 1-2.5 m)
  - Micropores are common and flow banding in micropores, leading to a flaky cleavage is often found, especially in tholeiite rock

- **Bottom scoria**
  - Main results 50-100 MPa  Q = 5-15
  - Main results for tuffaceous sediments < 1-40 MPa  Q = 2-4
  - Basalt lava sometimes with eroded surface  Q = 5-15
  - Fault breccias and dykes show lower rock quality

- **Bottom scoria, often 3-8% of the lava**
  - Typical thickness 0.3-1 m
  - Volcanic tuff interbed. Typical thickness 0.5-10 m
  - Typical grain size ranging from sandy and silty matrix, containing volcanic fragments up to 20 mm in diameter. Conglomerate is rare in the lower Eyjafjörður strata.
  - Eroded surface of underlying crystalline basalt, eroded surfaces are rare in the lower Eyjafjörður strata.
## Variable ground conditions

<table>
<thead>
<tr>
<th>Rock type</th>
<th>UCS (Mpa)</th>
<th>Youngs modulus (Gpa)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>avg</td>
<td>max</td>
</tr>
<tr>
<td><strong>Basalt</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Thleiite basalt</td>
<td>208</td>
<td>441</td>
</tr>
<tr>
<td>Porphyri basalt</td>
<td>134</td>
<td>305</td>
</tr>
<tr>
<td>Olivine basalt</td>
<td>143</td>
<td>303</td>
</tr>
<tr>
<td>Scoria</td>
<td>22</td>
<td>88</td>
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<tr>
<td><strong>Sedimentary rock</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Sandstone</td>
<td>37</td>
<td>117</td>
</tr>
<tr>
<td>Conglomerate</td>
<td>39</td>
<td>112</td>
</tr>
<tr>
<td>Siltstone</td>
<td>18</td>
<td>85</td>
</tr>
<tr>
<td><strong>Hyaloclastite/Moberg</strong></td>
<td>31</td>
<td>64</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Rock type</th>
<th>Typical</th>
<th>Q value</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Basalt:</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Low degree of jointing</td>
<td></td>
<td>5 - 10</td>
</tr>
<tr>
<td>Jointed basalt, silt-/claycoating on joints</td>
<td></td>
<td>2 - 5</td>
</tr>
<tr>
<td><strong>Scoria:</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Low degree of jointing, well consolidated</td>
<td></td>
<td>4 - 10</td>
</tr>
<tr>
<td>Poorly consolidated, clay filled</td>
<td></td>
<td>1 - 4</td>
</tr>
<tr>
<td><strong>Sedimentary rock:</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Altered tephra</td>
<td></td>
<td>0,5 - 1,5</td>
</tr>
<tr>
<td>Poorly consolidated</td>
<td></td>
<td>&lt;0,1</td>
</tr>
<tr>
<td><strong>Fault breccia</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Typical value</td>
<td></td>
<td>&lt;0,1</td>
</tr>
<tr>
<td>Wide zones + water leakage</td>
<td></td>
<td>&lt;0,01</td>
</tr>
<tr>
<td><strong>Dyke intrusions, columnar jointed basalt</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Minor water leakage</td>
<td></td>
<td>1 - 3</td>
</tr>
<tr>
<td>Water leakage and breccia accompanied</td>
<td></td>
<td>&lt;1</td>
</tr>
</tbody>
</table>
Weak rock – Thick sedimentary layers

- Thick layers of sedimentary rock
  - Thickness can vary from a few cm to be greater than the tunnel span
- Varying origin, tephra, siltstone, sandstone...
- Very low/low UCS
- Swelling properties
- Typical support:
  - Shotcrete with wire mesh
  - Rock bolts
  - RRS or Lattice girders with spiles
Weak rock – Thick sedimentary layers
Weak rock – Faults/fault breccia

- Faults vary greatly in character
- Minor faults
  - Narrow width, dm->m
  - Limited displacement
- Major faults
  - Several m wide
  - Filled with breccia.
- Minor faults generally non-problematic.
- Major faults collapsed
Weak rock – fault breccia, collapse
Excavation through collapse

- Exploration drilling
- Drilling of drainage holes
- Consolidation grouting
  - Chemical grouting in the fault zone above the crown
- Stabilization of debris/removal of debris, stepwise excavation to starting point of pipe umbrella
- Installation of Pipe umbrella
- Continued stepwise excavation of debris
Excavation through collapse
Weak rock – Altered rock

- Búðarhálsvirkjun
- 4 km headrace tunnel
- Span of profile 15 m.
- Altered olivine tholeiitic basalt
- Moderate alteration
  - UCS 30-90 MPa
  - E modulus 13-25 Gpa
  - Free swelling 85-140%
- Heavy support because of swelling properties of rock in a headrace tunnel
Water leakages

- GWT generally high
- Low permeability in tertiary basalts, in general
  - Low grade alteration
  - Secondary minerals
- Younger basalts can have medium or higher permeability
- Dike intrusions / faults / joints are water bearing
- Sedimentary layers, very low permeability/impermeable act as barriers
- Unexpected in-burst of water in spite of probing
- Has caused extensive grouting in some projects
Water leakages
Water leakages
Water leakages
Hot conditions

- Geothermal gradient in general 5-10°C/100m outside the active zone
- Encountered geothermal heat
  - Vadlaheidi ~ 65°C
  - Hvalfjordur ~57°C
  - Karahnjukar ~55°C
  - Husavik ~40°C
- Varying effects
  - Non problematic
  - Difficult working conditions
    - Heat
    - Humidity
Hot conditions

![Graph showing water flow, temperature, overburden, and gradient over tunnel length.]

- **Water flow**: Graph showing flow rate over tunnel length.
- **Water temp**: Graph showing temperature over tunnel length.
- **Overburden**: Graph showing overburden depth over tunnel length.
- **Gradient**: Graph showing geothermal gradient over tunnel length.

**Tunnel station (m)**

**Total water outflow (l/sec)**

**Water flow**

**Water temp**

**Overburden**

**Gradient**

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Icelandic Tunneling Society
Contributions

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Jón Haukur Steingrímsson
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Mannvit consulting Engineers
Icelandic Institute of Natural History, geological map
Vaðlaheiðargöng ehf.
Vegagerðin, The Icelandic Road and Costal Administration
TAKK FYRIR
Ground support practices and challenges in Finnish underground metal mines

P.J. Lappalainen, PL Mining Consulting, Finland
M.J. Lamberg, Pöyry Finland Oy, Finland

Abstract

There are 10 underground mines in Finland producing; copper, nickel and zinc in concentrates but also gold and chrome. Kittilä mine is the biggest gold mine and Kemi mine is the biggest chromite mine in Europe. All these mines are utilizing some modification of sublevel open stoping, with or without backfilling. Only highly mechanized ground support systems are applied – including scaling, combined bolting-meshing, shotcreting and cable bolting.

Ground conditions vary from very poor squeezing rock to rock burst prone very good rock type. Mechanized cable bolting has a decisive role in stope scale rock reinforcement - enhancing use of high capacity underground benching method even in poor rock condition. The first fully mechanized cable bolter was taken into use in Pyhäsalmi Cu-Zn-S mine in 1985. Important role of mechanized cable bolting in applying underground benching is shown by the paper. Annual amount of cable bolting has increased to level of 100-400 km in biggest mines.

In addition to safety aspects an objective in Ground support and reinforcement is also to reduce waste rock dilution and maximize mining recovery – i.e. economical aspects. Just minimizing of ground support costs is not targeted. Ground support is regarded more as an investment, a tool how to get mining process to be more safe and most profitable.

High lateral stresses are acting in ground. Stress control in mine and stope design is therefore vital, methods include designing stope geometries, stope sequencing, applying yield type pillars, fast-acting and dynamic bolts, de-stressing techniques and micro seismic monitoring. Empirical ‘trial and error methods’ were used in ground support dimensioning in the past. Currently rock mass quality (Q) determination with charts are as basis as in civil engineering. Numerical modeling with back analysis and observational methods are also applied. In the text future needs and challenges are discussed – how to handle ‘time factor’ reliably in design of deep mines with variable&unknown rock mechanics conditions? How to apply dynamic bolting and observational ground support dimensioning in practice?

1 Introduction

Most Finnish metal mines are sulfidic ore deposits (Figure 1), of volcanic or metamorphic origin, dipping quite steeply. Approximately 5% of land area is outcropping. Thickness of overburden is generally between 2m – 30m, and pre-Cambrian bedrock below principally is quite solid without thick weathered surface. The first 50-100m of the bedrock is more fractured. Variation of rock mass strength between mines is still quite large. Weakest rock is found basically in ultramafic ore deposits and in altered rock zones adjacent to ore bodies.
Ground support practices and challenges in Finnish underground metal mines

P.J. Lappalainen and M.J. Lamberg

Figure 1  Metallic ore mines in Finland 2015 (http://en.gtk.fi, Google Maps)

High horizontal stresses are acting in ground (Figure 2). Stress control in mine and stope design is therefore vital, methods include designing stope geometries, stope sequencing, applying yield type pillars, fast-acting and dynamic bolts, de-stressing techniques and micro seismic monitoring. Nowadays ground support methods: scaling, roof bolting, shotcreting and cable bolting are carried out by mechanized bolting jumbos. Aim from support mechanization is to ensure work safety, high ground support quality and capacity.

Figure 2  Horizontal and vertical in situ stress components in Finland (Compiled By Matti Hakala, Stress Measurement Company Oy)
The biggest underground mines – Kemi Cr, Pyhäsalmi Cu-Zn-S, Kittilä Au, Kylylahti Cu-Zn-Co and Pampalo Au- mines, are utilizing similar type underground benching (UGB) method with delayed backfilling and cable bolting. The above-mentioned mines apply transverse UGB in thick parts and longitudinal UGB in thinner parts, progressing ‘bottom-up’, utilizing mainly down-hole production drilling and remote controlled ore LHD via bottom drive (Figure 3). Level spacing in these mines varies between 20-40 m. In addition to high productivity an advantage in the UGB is that the amount of needed mine development –i.e. tunnel drifting- is reduced due to the fact that no separate draw points are needed for mucking. Design is also simple, UGB stope lengths can be varied based on stope stability needs. A disadvantage in UGB method is that stope backs must be very stable – resulting in very heavy systematic cable bolting ‘skin-support’ of ore drifts, at least in poor rock conditions (Figure 4). In general the cable bolting skin support means a combination of plated cable bolts, mesh-reinforced shotcrete and roof bolts. In Kemi mine the total amount of cable bolting exceeded 400 km in 2015, carried out by 5 mechanized cable bolting jumbos. The amounts of ground support in the biggest underground metal mines are shown in table 1.

![Figure 3](image.jpg)  Principle of underground benching with delayed backfilling, Pyhäsalmi Mine (Bergström et al. 2014)

<table>
<thead>
<tr>
<th>Mine</th>
<th>Cable Bolting (1000 m)</th>
<th>Roof bolting (pcs)</th>
<th>Shotcreting ( m³)</th>
<th>Ore (1000 t)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Kemi</td>
<td>408</td>
<td>138,000</td>
<td>32,000</td>
<td>1,951</td>
</tr>
<tr>
<td>Kittilä</td>
<td>200</td>
<td>85,000</td>
<td>18,000</td>
<td>1,484</td>
</tr>
<tr>
<td>Pyhäsalmi</td>
<td>39</td>
<td>21,000</td>
<td>6,000</td>
<td>1,378</td>
</tr>
<tr>
<td>Kylylahti</td>
<td>29</td>
<td>23,000</td>
<td>5,500</td>
<td>720</td>
</tr>
</tbody>
</table>

Table 1  Ground support amounts in the biggest underground mines in Finland 2015
Ground support practices and challenges in Finnish underground metal mines

P.J. Lappalainen and M.J. Lamberg

2 Ground support in mine development and drifting

Careful drilling and blasting procedure with scaling is a basis to prevent unnecessary ground support. Modern “data” jumbos are therefore currently used in drilling to follow designed drilling pattern, contour hole spacing in smooth blasting and rounding of wall-floor corners i.e. also proper mechanized scalers equipped with ‘scraper’ head or hydraulic hammer. Empirical rock mass quality Q-system and MRMR-system or local empirical design system is used for ground support dimensioning with different ground support recipes, depending on rock mechanics conditions and drift types (Figure 5).

Figure 4 Cable bolt skin support of stope roof with mesh and shotcrete. High deformations at walls and roof (photo by M. Lamberg)

Figure 5 Ground support design recipes example, cable bolting profile from Kittilä Mine
In roof bolting fast acting bolts, cement grouted Kiruna bolts or resin anchor bolts are applied in permanent tunnels, in decline, repair shops e.g. In short term drifts inside the orebody, friction bolts, Swellex and split set bolts are used. Mechanized installation is applied to bolting and meshing (Figure 6), as in most mines, manual bolt installation below 500m level is not allowed at all to guarantee work safety. Dynamic D-bolt is becoming popular due to its capability to withstand high deformations in soft rock and to allow sudden movements in rock burst prone rock without breaking. Cable bolts are used when longer bolts are needed for example in tunnel intersections or in civil infrastructure caverns.

Figure 6  Mechanized mesh & bolt installing at Kemi mine using jumbo (photo by M. Lamberg)

Most drifts are shotcreted after mucking and scaling. Technology development in shotcreting (concrete spraying) during the last decades has been impressive (Figure 7). Higher concrete strength and setting characteristics with low water cement ratio (W/C) have been attained with new chemicals. So that 5 cm concrete layer can be sprayed in one stage without extra rebound e.g. reducing tunneling cycle time.

![Figure 7](image.png)  Impact of low water/ cement ratio to shotcrete quality  (J.Lehto 2014)
3 Cable bolt ‘skin support’ in underground benching

Rock reinforcement by cable bolting has made high capacitive underground benching feasible even in poor rock conditions. The first mechanized hydraulic cable bolter was developed between Tamrock Oy and Pyhäsalmi mine and was taken into use in 1985. Currently two boom cable bolting jumbos are used in many Finnish mines, manufactured by Atlas Copco or Sandvik (Figure 8). The Jumbo operator can drill and cement grout the hole, starting from hole bottom and push steel strand with any length into hole with the bolter. Cable reel of +500 m strand is mounted into the Jumbo and it can cut hydraulically strand during the installation procedure at desired length. Normally the designed bolt length varies between 5 m and 10 m, comprising of one or two 15.2 mm diameter seven-wired steel strand cable per hole, as required. Surface plates are installed later by wedge barrel type surface anchors. Twin-strand cable bolt withstands load up to 500 kN.

Advantages in mechanized bolting are good quality control with low water cement ratio and working safety, when compared to manual cable installation procedure. Also the hole plugging phenomena in weak rock can be minimized because hole drilling, cement grouting and cable installation are carried out in the same working cycle. However, most important is that by mechanized cable bolting rock pre-reinforcement around stopes can be done with variable bolt lengths as desired from small development drifts, before actual stoping, i.e. before large rock movements. Also various design principles – ‘hedge hog’ bolting or sub-parallel ‘mandolin’ bolting can be utilized – in combination with surface support by mesh-reinforced shotcreting and roof bolting (Figure 9). Already 30 years ago these rock support designs with yield pillars were utilized in Pyhäsalmi Mine successfully.

Figure 8 Mechanized cable bolting by two-boom cable bolting Jumbo in Kemi Mine, (Photo by A. Frant)
Figure 9  a) Principle of modified sublevel stoping in Pyhäsalmi in the 80’s. Roman numbers show the blasting sequence of the stope. (Lappalainen and Antikainen, 1987)  b) cable bolting pattern for a sublevel stope, with both transversal and parallel bolts installed. The parallel “mandolin” bolting were to slow down the wall rock bulging. (Lappalainen and Antikainen, 1987)

Ground support and stope design are to be combined (Figure 10). Orebody outlines should be determined accurately, with reasonable infill drilling density. Otherwise ore losses and waste rock dilution will increase, resulting lower profitability. Often dilution is not accurately taken into account when estimating the production efficiency of the UG mine (Lappalainen and Pitkäjärvi 1996). If ore development drifts in longitudinal benching will undercut hanging wall waste rock ‘beam’, it will be in practice impossible to eliminate dilution by ground support (Figure 11). Individual stope shaping and design streamlining according to rock stresses is essential – to allow horizontal compressions go through. Transverse UGB stopes are to be oriented along maximum horizontal in-situ stress if possible, hence overall stope stability is enhanced (Bergström et al. 2014, and Sahala 2016). Slightly yielding pillars between stopes improve stability essentially. In most of the Finnish mines utilizing UGB, cemented rock filling (CRF) is used as backfill material in primary transverse stopes. However, paste backfilling is preferred to day due to better fill tightness quality, which is needed to reduce the mine wide rock deformations (Bergström et al. 2014).
Figure 10 Transverse underground benching in Kemi Mine; example of stope design actual stope boundaries in poor rock conditions and ground support design (Lappalainen et al. 2015)

Figure 11 a) External dilution from a hangingwall failure in a large underground bench stope (Villaescusa 2014). b) External dilution from a hangingwall failure in underground bench stope due to undercutting of hangingwall (ore development drive in waste rock side) (Andersen and Grebenc 1995)

4 Numerical simulations for mining

Numerical rock mechanical simulations have been used as a basic forecasting tool for underground mine design purposes in many of the Finnish underground mines. The numerical simulations evaluate the rock mass behavior, (deformations, displacements and stress peaks) after excavations in the mine and in the vicinity of the mine (Figure 12). Evaluation of the future deformations in the rock mass are then used to adjust excavations and ground support to more suitable design in order to maximize production efficiency and stability of the mine. Stability problems in the mine will often lead to minor or medium size production losses (dilution and ore loss). Stability problems may also cause a working safety issues at the tunnels. In major stability problems the production cut off can have a huge economic impact to the mine and eventually may stop of the whole mining operation.
Figure 12 A simulation results showing +30 cm displacements with simulated vertical and horizontal pillars. Simulation was performed using 3DEC software.

In underground mines the in situ rock stresses and rock mass properties are key data inputs when estimating the long term stability of mining. Numerical simulations require at least an estimate of the in situ rock stress and rock mass parameters in order to complete reasonable reliable forecasts for production. The level of the initial data quality and uncertainty depends from the phase of the mining project. Pre evaluations, scoping and prefeasibility study phases, are often limited to very little or sparse initial geotechnical and rock stress data and hence the uncertainty in the numerical simulations is considered high. In the later project phases, the required accuracy in the numerical simulations is higher, meaning that the input to the simulations should be more accurate and consider regional geological differences in the parameters.

The in situ rock stresses are generally determined using varying measurement techniques from drill holes or directly from tunnels, overcoring methods, LVDT e.g. (Hakala and Valli 2016). The rock mass properties are determined by core logging, tunnel mappings and rock strength estimations in a laboratory. Rock mass quality estimation in the mine can be done in using variable international mapping methods i.e. Q-method, MRMR and GSI.

Numerical simulations have been used for following purposes in mine design at Finnish mines:

- Estimating the maximum size of the underground openings: Stope sizes, tunnel profiles and the size of the infrastructure caverns in the mine.
- Evaluating the demand for ground support in the various locations in the mines: For example cable bolting in the production stopes and tunnel support.
- Forecasting the future production schedule in the mine. Long term stability of the mine and local stability issues.
- Back calculate previous events or to calibrate models to measured deformations in the mine.

For ground support design, simulations generally require some understanding of the previous conditions in the mine environment in order to calibrate the rock mass behavior in the models. Often it is more useful to evaluate the ground support requirements for different mining areas rather than focusing into detailed support patterns. For detailed ground support pattern design simulations rarely give any better results than empirical and practical evaluation of the support needs.

Regional simulations can give a hint for the mine design to target more heavy support into certain areas of the mine. In the Figures 12 and 13 is a seen regional simulation to forecast overall stability of the Kemi mine.

![Figure 12](image-url)

Figure 12  A Regional mine simulation at Kemi underground mine to evaluate future areas of high displacements and to target more ground support for such areas. Simulation was performed using 3DEC software
When considering numerical simulation methods for evaluation of the capacity of the underground excavation and ground support system, following factors should be taken into account:

- The excavation geometry. More details require longer calculations times. Consider simplified geometries for initial estimations. When estimating the capacity of a ground support system simplified models may be suitable if geological environment is considered uniform.

- The complexity of the rock mass in many mines requires special attention when evaluating the regional stability of the mine using numerical methods. Simplification of the geology is always needed at certain range. Structures in rock mass should be included into the modelling correctly and explicitly to capture the realistic response of the ground support system.

- Failure criterion of the potential instability should be determined prior to modelling. The failure criterion depends on many factors and the type is not always clearly defined in preproduction phases.

- Pre-existing excavations or mining nearby can have effects to the ground support system of the new excavations. The rock mass around the new excavations can be already yielded or subjected to stress loading/relief.

In the Finnish underground mines, numerical simulations have been used in varying conditions, ranging from squeezing rock masses to very brittle rock mass behavior hence the numerical simulations are always adapted to the conditions in the mine in questions. In the Kemi, Kylylahti and Pyhäsalmi underground mines the simulations are used in multiple forecasting and mine design purposes. The simulations have revealed for example issues in the underground mine design. Based on the simulation results the secondary stresses caused by mining can be taken into account when permanent structures are being designed and built (Lamberg et al. 2016). The type and need for rock support should be estimated already in the investment stage of a new project. The improvements adapted from simulation results have been considered relevant many cases and the changes in design have improved functionality, efficiency and safety of the mines.
It is clear that as technology develops in other areas of the mining business, the numerical simulations should develop as well. Hence lots of research has been conducted to maintain world class knowledge in the area of rock mechanics and numerical simulations in Finland.

5 Future needs and challenges

While underground mining goes to deeper levels, both in-situ and secondary stresses due to stoping will increase, causing stability problems. Following increase in ground support costs. Also the same ground support methods, which were good near surface levels can’t be applied in deeper levels. Time dependent inelastic behavior of rock is to be taken into account and ground support has to withstand high deformations without breaking. Deformations can be slow and continuous in squeezing rock type or they can be sudden in hard burst-prone rock. Energy-absorbing, Dynamic yield type bolts – D-bolts e.g - are then to be utilized instead of stiff bolting (Li et al. 2014). Advantages in D-bolt are its simplicity and its high load capacity, both in static and in yield stage (Figure 14). Both resin and cement grouting can be used (Näsi and Harju 2016). Yield type pillars are also to be designed. Mesh-reinforced shotcrete can ‘follow’ much higher deformations than fiber-reinforced shotcrete and is therefore preferred. In stope support systematic ‘skin cable bolting design’ – combination of plated cable bolts, mesh-reinforced shotcrete and roof bolts - along ore drives are applied. To allow high cable bolt deformations without breaking the part of cables should be debonded – leaving e.g. the first 1.5 m of cables without grouting, as done in Kemi mine. In future the cable bolts are possibly ‘plained’ piecewise to adjust cable bond strength according to needs.

![Figure 14 D-bolt principle with anchoring points (J.Lehto 2014)](image)

In high stress environment a big challenge in numerical simulation is how to handle time factor effects and heterogeneous rock mass behavior realistically in 3D. Even when using sophisticated plastic numerical codes and especially when geological input data in details is always unknown, as well as rock stress distribution. Effects of fault zones are difficult to predict (S.D. McKinnon, RS 2016). Therefore empirical and observational methods in ground support engineering are applicable and widely used (Syrrjänen et al. 2016). Trial and error method for design is viable, if rock mass behavior and ground support ‘functioning’ together is properly understood. And empirical feedback data of rock mass behavior is systematically collected and analyzed by geotechnical real-time monitoring (extensometers, stress cells, microseismic network etc.) and by visual observations (Figure 15).
bolting emphasis should be put more on simple load monitoring devices and convergence measurements by scanning, to get statistically enough rock mass & ground support information. Aim in deep mining is not to prevent rock breaking because it is impossible. Aim is to prevent uncontrollable caving and therefore Yield type stope & pillar design and ground support design will be needed more and more.

![Figure 15](image-url) Wall deformations in drift pv116op1-350 in Kemi mine between 4.3.2015 and 19.2.2016. Between the measurements the profile area has decreased by 7.1 m²

It is well known that some part of ground support will be ‘extra’ or additional to ensure production reliability and safety. Especially when using high capacity mining method in poor rock conditions, as in benching method with delayed backfilling and remote controlled loading via ore ‘stope’ drifts. However, it is in practice impossible to determine exactly which cable bolts e.g. are extra and could be left out without risking stope stability due to geologic uncertainty factors. In that sense mining is deterministic, not probabilistic by nature. Geotechnical ground support is a natural part of mining process, targeting to increase safety and mine production reliability & quality. One stope failure quite easily costs as much as many months’ ground support costs due to increased ore losses and waste rock dilution. So emphasis in ground support design is quality and total optimization – not just focusing on ground support costs e.g. Ground support should be comprehend more as an investment. Actual ground support design is a continuous learning process.

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The Stockholm bypass project – past, present and future

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Abstract

This paper gives a summary of the major motorway project Stockholm bypass history and information of some of the ongoing contracts. The description of the past included the earlier investigation regarding an outer bypass, Västerleden, of Stockholm. The process from feasibility study to Land acquisition plan is presented briefly. The paper also handles the introduction to the Stockholm bypass regarding the tunnel system and planned contracts. The introduction follows by a short description of the detailed design of the rock tunnels that has been done in the project. In the detailed design BIM has been introduced and the advantage are pointed out in the paper. The ongoing contracts that are described in the paper are FSE210 Access tunnel Skärholmen, FSE607 Access tunnel Akalla and FSE430 Rock tunnels Johannelund. In the end of the paper information is given on what will come in the near future.

1 Introduction

The Stockholm bypass is a 21 km long motorway west of Stockholm, Sweden, and is intended to replace the aging motorway system going through the city, see Figure 1. It will improve substantially the local, regional and national traffic requirements.

Figure 1 Overview of the Stockholm bypass

To reduce the footprint on environmentally sensitive land, the Stockholm bypass is going mainly through a total length of 50 km of tunnels including two main tunnels with three traffic lanes in each, underground junctions and access ramps.
The main traffic tunnels are passing beneath Lake Mälaren at three locations and at a maximum depth of 88 m below sea level, see Figure 2.

![Diagram of the Stockholm bypass and passages beneath Mälaren](image)

**Figure 2**  Profile of the Stockholm bypass and the passages beneath Mälaren, shown with red ovals

## 2 The history of the Stockholm Bypass

Since Stockholm is situated on several islands there is always a challenge to improve the transport communications through the city and the region. A variety of solutions to improve the north-south transport links have been studied.

The latest link that opened for traffic across Lake Mälaren was in the late 60’s when the E4 Essingeleden opened for traffic. Since then The Swedish Transport Administration (before 2010, Swedish Road Administration) has studied several alternatives to improve the communications from south to north.

In the beginning of 1990 an outer bypass was investigated called Västerleden. Västerleden had almost the same alignment as the present Stockholm bypass. The main differences to the current bypass scheme is that the Västerleden ended at Bergslagsplan and run on bridges when crossing the three water passages, see Figure 3. In 1996 the Land acquisition plan was completed, but then the financial conditions changed and the detailed design of Västerleden never started.

![Comparison of Västerleden and Stockholm bypass](image)

**Figure 3**  Västerleden, the blue lines indicate the road above ground, is shown in the left figure. The Stockholm bypass is shown in the right figure for comparison. The dotted lines indicated the road in tunnel
After several years the Swedish Road Administration received a new assignment to perform a feasibility study of a link between the northern and the southern part of Stockholm. The purpose of the feasibility study was to find answers to a four step principle. The first step was to affect the transport demands and choices. The second step was to have a more efficient use of the existing infrastructure. The third step was to make reconstruction on the existing infrastructure and the fourth step was to construct new infrastructure.

The feasibility study was finished in 2001 and that a new road solution most favorable, which resulted in work starting on a preliminary design plan. During the preliminary design three possible corridors for the north–south link were investigated; the Stockholm Bypass, the Ulvsunda Diagonal and the Combination Option (Norberg et al. 2005), see Figure 4.

The Swedish Road Administration suggested The Stockholm bypass as the leading alternative when they in 2008 submitted a report to the Government regarding consideration of permissibility. In September 2009 the government decided to give the motorway project the go-ahead (Trafikverket, 2016).

In the preliminary study The Stockholm Bypass runs mainly through tunnels except at the crossing over the water between Lovö and Grimsta and over E18 at Hjulsta where the road goes by bridges.

The next phase of the project was to do a conceptual design which results in a land acquisition plan. The land acquisition plan contains technical drawings of the road along with drawings showing the areas of land which will be needed. Since the project has a major impact of the environment the land acquisition plan was submitted together with an environmental impact assessment (EIA). In the spring of 2014 the land acquisition plan of the Stockholm Bypass acquired legal approval.

Parallel to the land acquisition plan The Swedish Transport Administration applied for permits from the Land and Environment Court. The applications concerns permits to allow limited groundwater lowering from the rock tunnels and to build and operate three temporary harbours to allow the

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**Figure 4** Three possible road corridors in the preliminary design plan
excavated rock to be transported from the site using barges. The permits were given to the project in December 2015.

During the conceptual design phase the bridge across the water between Lovö and Grimsta was replaced by a rock tunnel. The Stockholm bypass now runs in tunnels from Kungens kurva in the south up to Hjulsta in the north, a length of 17 km. This will make The Stockholm bypass the largest road tunnel in Europe.

3 The Stockholm bypass

As shown in Figure 1 above, The Stockholm Bypass runs from Kungens Kurva to Häggvik through tunnels and over a bridge. Along the new E4, six new traffic interchanges will be built to give access and exits to the new motorway. These six interchanges are situated in Kungens kurva, Lovö, Vinsta, Hjulsta, Akalla and Häggvik. In the south separate ramps will be built to enable bus traffic to and from Skärholmen.

The main road consists of two tunnels that separate the traffic in each direction, see Figure 5. Each main tunnel has three lanes of a width of 3,5 m and an installation culvert, signs and fans. The outer dimension of the main tunnels is 16,3 m and the inner dimension in the traffic area is 13,5 m.

![Figure 5 One of the main tunnels and its different components](image)

Even if the distance from south to north is about 21 km a total of 50 km rock tunnels will be excavated. The main tunnels are 18 km long and the ramp tunnels sum up to about 14 km rock tunnels. Additional to the traffic tunnels there are also seven access tunnels. These access tunnels are situated in Skärholmen, Sätra, southern Lovö, northern Lovö, Johannelund, Lunda and Akalla.

The excavation method is drill and blast and about 19 million tonnes of rock will be transported mainly through the access tunnels. At three locations the rock masses will then be transported by ships and barges instead of using lorries on public roads. The rock mass is transported from the access tunnels by a conveyor belt to each temporary harbour, see Figure 6.
Even if the Stockholm bypass is a large tunnel project there are also major geotechnical challenging at the traffic interchanges. As the schematic illustration shown in Figure 7 indicates there are challenging ground conditions due to a combination of hard rock, moraine, loose clay and ground water. From the ground surface the road will run in an open concrete cutting followed by a concrete tunnel before entering the rock tunnel. During the construction phase high rock slopes and sheet piles or other ground support methods will be used. Due to permits from the environmental court there are also requirements that the ground water level cannot be lowered more than 0.3 m.

All construction works are divided into a large number of contacts in order to have a short construction time as possible.

4 Detailed design

The detailed design of a large project as The Stockholm Bypass has been divided into several design contracts. The contract for detailed design of the rock tunnels (FSKO2) was assigned to ÅF/Scott Wilson (currently ÅF/AECOM) in 2010. The contract was appealed and therefore the start of the detailed design didn’t start until September 2011.
The design contract for FSK02 is responsible for detailed design (client design) of the rock tunnels, rock reinforcement and grouting etc. This means that the consultant has to execute detailed design drawings and specifications put together in eight different construction tenders. For a project of this size, it would represent a tremendous number of drawings showing plans, profiles and sections. Therefore, The Swedish Transport Administration decided that the project will use building information models (BIM or VDC) both in the detailed design phase and the construction phase. BIMs have therefore been developed to use for visualization, estimating quantities and surveying.

### 4.1 BIM

All design work is hosted by the software ProjectWise developed by Bentley Systems, Inc. The software is accessible to all the members of the project. Due to the complex structure of information in the BIMs, all BIMs in The Stockholm Bypass are always associated to their RFA. RFA stands for Redogörelse För Anläggningsmodell in Swedish and is basically a user manual for the BIM that explains how the model has been set up, its use, its accuracy, its limitations etc. (Lindström & Outters, 2014).

This is the first time The Swedish Transport Administration has chosen to use BIM in such a large project and the learning curve has been steep. Today, the advantage of using BIM during the design phase has been many. For examples the review process been speeded up and the quality of the produced BIMs and documents is noticeably higher than for a traditional 2D-CAD based design. There has also been easier to identify collisions between different technical areas, for example between installation and rock. The final advantages of using BIM instead of traditional drawing and 2D-models will hopefully be seen during the construction phase.

### 4.2 Design of rock reinforcement

The typical design of the reinforcement is made up for standard tunnelling conditions, i.e. rock mass has a Q>0.1 and when the tunnel span is less than 21 m and the rock cover is more than half the tunnel span. For other conditions the reinforcement is individually designed.

When designing the typical reinforcement, the rock designer assumed a preliminary reinforcement based on the conceptual design, Barton’s Q-system and some relevant reference projects. The preliminary reinforcement is then optimized and verified by analytical calculations and an unsupported numerical model.

The typical reinforcements for the main tunnels are shown in Figure 8.

<table>
<thead>
<tr>
<th>ROCK CLASS</th>
<th>ROCK QUALITY</th>
<th>ADHESION REQUIREMENT BETWEEN SHOTCRETE AND ROCK, (MPa)</th>
<th>BOLT</th>
<th>FIBRE REINFORCED SHOTCRETE</th>
<th>REINFORCEMENT CLASS</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>SPACING, (m)</td>
<td>LENGTH, (m)</td>
<td>THICKNESS, (mm)</td>
<td>WALL</td>
</tr>
<tr>
<td>I</td>
<td>Q &gt; 10</td>
<td>0.5</td>
<td>S*</td>
<td>5</td>
<td>50</td>
</tr>
<tr>
<td>II</td>
<td>4 &lt; Q ≤ 10</td>
<td>0.5</td>
<td>2.0*</td>
<td>5</td>
<td>4</td>
</tr>
<tr>
<td>III</td>
<td>1 ≤ Q ≤ 4</td>
<td>0</td>
<td>1.7</td>
<td>5</td>
<td>4</td>
</tr>
<tr>
<td>IV</td>
<td>0.1 &lt; Q ≤ 1</td>
<td>0</td>
<td>1.5</td>
<td>5</td>
<td>5</td>
</tr>
</tbody>
</table>

*If the adhesion requirement not can be fulfilled the reinforcement shall be adjusted to current conditions, decision about measures will be decided by the client with pattern bolting, S = 1.5 m, as a starting point.

Figure 8 The typical reinforcement for the main tunnels (taken from drawing, 000B2403)

### 4.3 Design of grouting

The environmental court has given permits how much the ground water the project can be drain both during the construction phase and after the project is finished. This means that the design of the grouting is important.
The design of the grouting has resulted in three different grouting classes, A, B and C. Grouting class A is used in areas with the requirement on conductivity, $K_{inj} \approx 1 \times 10^{-8}$ m/s. Grouting B is used in areas with requirement on conductivity, $K_{inj} < 1 \times 10^{-8} - 5 \times 10^{-9}$ m/s. Grouting C is used in areas with even higher requirement on water tightness, $K_{inj} < ca 5 \times 10^{-9}$ m/s.

The fan in grouting class A, see Figure 9, is half the amount of boreholes as the fan in grouting class B, see Figure 10. In areas where grouting class A and B are forecast, every second borehole in the fan is drilled. While the client evaluates the results from the initial boreholes using MWD, Measurement While Drilling, the rest of the boreholes in the fan are drilled. Before the boreholes in the second round are drilled, the contractor will get information from the client if additional boreholes are required. The contractor then starts the grouting process when all the boreholes, including additional boreholes, are drilled.

In areas where grouting class C, see Figure 11, is forecast, no evaluation of the results from MWD is made.

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**Figure 9** The fan for Grouting A (taken from drawing 000B2A01)

**Figure 10** The fan for Grouting B (taken from drawing 000B2A02)
4.4 The design responsibilities for FSK02 during the building phase

In the design contract, FSK02, the responsibilities for the design are not complete until all the rock tunnels are excavated. In the construction phase the design responsibility is finalized through the work done by the geologists. The geologists are mapping the rock surface in the pre-cut and in the rock tunnels. After the mapping, the geologist determines the final rock reinforcement that is required according to the typical reinforcement classes. The geologist also evaluates the data from MWD and provides recommendation for drilling additional boreholes for grouting.

5 Construction phase

At present three construction contracts for rock tunnels are ongoing, FSE210 Access tunnel Skärholmen, FSE607 Access tunnel Akalla and FSE403 Rock tunnel Johannelund. There are also three construction contracts for the traffic interchanges assigned to FSE105 Concrete tunnel Kungens kurva, FSE61 Interchange Akalla, FSE62 Interchange Häggvik.

A short report from the three rock tunnel contracts is given below.

5.1 FSE210 Access tunnel Skärholmen

This client design contract was signed with Subterra/STI in January 2015. The contract consists of excavation of two access tunnels, one in Sätra and one in Skärholmen. In order to get access to the access tunnel in Skärholmen, parts of the future bus ramps in Skärholmen is excavated. The contract also includes building the temporary harbor in Sätra.

In Skärholmen, the design of the open cut and the access tunnel had to be redesigned in the autumn of 2015 due to changed geological conditions. The rock mass consists of large unfavorably oriented graphite filled weakness zones, see Figure 12.
The northern rock slope in the open cut was originally designed as an almost vertical slope of 10:1 but due to the more unfavorable rock condition the redesign was to excavate a shallower slope with an angle of 45° and with a backfill instead of using rock bolts as reinforcement, see Figure 13.

The weakness zone that was identified in the open cut continued into the rock tunnels leading to installation of reinforced shotcrete ribs as reinforcement. The subproject Tunnel South determined to move the start of the access tunnel to avoid the zone and to avoid even more heavy reinforcement in the junction of ramp tunnel and the access tunnel. The access tunnel was moved about x meter in the bus ramp.

However, in Sätra, the geological conditions were better than predicted which made an earlier start of the rock tunnel possible with 5 meters (Figure 14).
Figure 14 Excavation of the access tunnel in Sätra.

The excavation of the tunnels is now up to speed, and in the beginning of June, 400 m of totally 900 m of the contact have been excavated by the contractor

5.2 FSE607 Access tunnel Akalla

This client design contract was signed with Veidekke in October 2015. The contract consists of excavation of one access tunnel. The pre-cut and the beginning of the access tunnels coincide partly with the existing tunnel down to facility of Fortum Värme, see Figure 15.

Due to worse rock conditions than expected in the area a redesign is undertaken including an increased inclination of the ramp in the pre-cut, deepened and reinforced sheet piling, a widened and extended pre-cut as well as a new reinforcement in the beginning of the rock tunnel. As a result of the new design and the extended pre-cut the work area are expanded about 30 meters along the tunnel alignment. The new design will be finalized early summer 2016. However, the extensive redesign will not result in delays of the construction schedule.
5.3 FSE403 Rock tunnel Johannelund

This client design contract was signed with a joint venture between Implenia Construction and Razel-Bec in August 2015. This is the first main tunnel contract that is signed in the project. The contract consists of excavation of two main tunnels and four ramp tunnels, see Figure 16.

Additional to the tunnels for traffic, evacuation tunnels, technical utility space and two air exchange stations, one in each direction, are also included in the contract. In total, about 1 500 000 m³ rock are to be excavated in the contract.

The contractor has started with the pre-cut to the southern ramp tunnels but due to low rock cover the opening of the rock tunnel is moved further in.

The same ramp tunnels pass under Lövstavägen after 140 m from the opening of the rock tunnels. During the design phase, the investigation showed that there is not any rock cover over a distance of 30 meter, and low rock cover over a distance of 25 meter, see Figure 17. In order to stabilize the ground, jet grouting is ongoing at the moment, see Figure 18. The final reinforcement will be made by a concrete tunnel.
At present there are five more contracts for rock tunneling waiting to be contracted. The two contracts at Lovö, FSE302 Rock tunnels Norra Lovö and FSE308 Rock tunnels Södra Lovö have been appealed during the spring and the project is currently waiting for the outcome from the Administrative Court of Appeal.

In the south, the subproject South received the tender in the end of May for FSE209 Rock tunnels at Skärholmen, and evaluation of the bids is ongoing. The aim of the project is to have an assigned contract in the end of June and to start the construction works in the beginning of January 2017.

Subproject Tunnel North is responsible for four different rock tunneling contracts. Except for FSE403 and FSE607 which are presented above, they are currently evaluating the bids for FSE410 Rock tunnels.
Tunnels at Lunda which were sent in early in June and they are also preparing to send out the tender documents for FSE613 Rock tunnels of Akalla in October 2016.

In the ongoing contract FSE403 Rock tunnels Johannelund is the first main contract that will give result how successful the use of BIM is instead of using a large number of traditional drawings.

Acknowledgement

First of all, I would like to thank all of my colleagues in the project Stockholm bypass project. It is a pleasure to work with such friendly and experienced people. And then of course, Professor Erling Nordlund at Luleå University of technology, who asked me to be an invited speaker at Ground support 2016.

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Rock support in the Boliden Mines

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Abstract

Boliden is a Swedish metals company that has mined more than 50 mines since the first mine in Boliden was opened. The ore bodies have mostly been tabular, sub-vertical, massive sulphide deposits with relatively small tonnages. Due to the rock conditions and ore geometries, Cut and Fill mining has been the preferred mining method in the majority of the mines. As of today, also long hole stoping is used in a couple of mines.

The policy of the company is to eliminate the risk for accidents caused by rock falls and the authorities require that the planning of the work results in a safe workplace. Such planning requires knowledge about the rock conditions, how an excavation affects the stress distribution in the vicinity, failure modes and failure patterns for the cases when the load exceeds the rock mass strength in different geological conditions and about the function of different rock support subjected to different loads and in different geological environments.

This paper presents the main rock mechanical problems for different mining methods and geological conditions together with how the rock support is used to maintain safety and stability of openings while securing high productivity in spite of the relatively small scale of the mining.

1 Introduction

1.1 Boliden – the company

The Boliden Mines:
1. Aitik
2. Kevitsa (Finland)
3. The Boliden Area: Kankberg, Renström, Maurliden and 4. Kristineberg
5. Kylylahti (Finland)
6. Garpenberg
7. Tara (Ireland)
8. Laisvall (closed 2001)

Figure 1 The localisation of the mines operated by Boliden and described in the paper

Boliden is a metals company with focus on sustainable development. Our roots are Nordic, but our business is global. The company’s core competence is within the fields of exploration, mining, smelting and metals recycling, the localisation of the mining areas are shown in figure 1. Boliden has a total of around 4,900
employees and the annual turnover 2015 was SEK 40 billions. Its shares are listed on NASDAQ Stockholm, segment Large Cap.

1.2 The mining

Boliden has operated more than 50 mines since the 1920’s and has gained experience from a very wide span of ore geometries, rock conditions and applied technologies. The varying ore geometries and rock conditions have resulted in the use of different mining methods and a quite large difference between the requirements on the rock support between mines and ore bodies. This paper will describe how the requirement on the rock support varies with mining methods and rock conditions within the Boliden mines.

2 Geology and mineralizations

Historically and currently most of Boliden’s underground mines have been volcanogenic massive sulphides deposits. In these mines the mineralization’s are the result of volcanic activity, which combined with hydrothermal deposition of metals causes alteration of the original rocks. Later tectonic activity causes folding and shearing resulting in both plastic and brittle structures. All of this can result in the occurrence of weak schists. The typical ore is tabular and sub-vertical, 5-20 m wide and made up by massive or semi-massive sulphides while the contact zones often consist of sericite, biotite, chlorite and even talc schists, figure 2. A few larger mineralizations have been and are being mined. The largest is Aitik, a large porphyry copper deposit open pit mine.

![Ore profile showing example of ore geometry and chlorite schists in the contact zone.](image)

Figure 2  A vertical profile through the ore showing an example of ore geometry and the occurrence of chlorite schists in the contact zone. Red line along the drill hole = massive sulphide, green = chlorite schist, RQD presented as blue blocks for each meter along the drill holes. The numbered squares are planned drift & fill stopes.

3 Mining methods and Rock mechanics

3.1 Mining methods used in Boliden

The combination of the ore geometries and the rock conditions in the ore bodies historically and currently mined by Boliden has very often resulted in choosing some alternative of Cut and Fill mining. The method is very flexible as the scale of the method can be adapted to the geometry and rock conditions and it is possible to follow the sometimes challenging changes of the ore boundaries. Combinations of Cut and Fill and benching methods has been used to a lesser extent but in larger ore bodies long hole stoping methods has been implemented. In thin, horizontal, mineralizations and good rock conditions Room and Pillar mining has been the obvious choice but also in combination with backfilling in wider subvertical ore bodies. Open pit mining in different scales is also used.
3.2 Cut and fill

Figure 3 shows how cut and fill mining can be used in different scales depending on the rock conditions and ore geometry. It is a very flexible method allowing high ore recovery in complex geometries (Kolsrud 1983). A slightly larger scale is possible by using the Avoca method, figure 4 a) while figure 4 b) shows a common method of mining sill pillars (the horizontal pillar between levels) by using vertical drill holes, retreating towards the entrance of the stope.

The implementation of Cut and Fill requires knowledge about the requirements of the properties of the fill, such as permeability and strength, and of the implications of different mining sequences when mining several drifts on the same level. For the actual mining of the stopes the choice of and design of the rock support is a very important issue as it influences both stability and safety.

The largest influence on the mining conditions relates from the mining induced stresses as shown in figure 5. As mining develops the stress concentration in the backs increase and as the distance towards the upper mining level is decreased, the pillar between the stope and the bottom of the upper level will be subject to high stresses. Eventually this leads to failure, either of the surrounding rock, the contact zones or the pillar itself. Depending on the rock conditions the failure can be slow, leading to large deformations, or more
brittle causing seismic activity, in the worst case in the form of rock bursts. Figure 6 shows a typical face in a Cut and Fill stope where the high concentration of stress has caused failure of the rock in the face.

![Figure 5](image1.png)

**Figure 5** Development of stress shown in a profile through a typical subvertical, table shaped ore body mined with cut and fill mining. The stress concentrations in the back of the stopes increase with the height of the extraction and develop to even higher levels in the sill pillar towards the mined out stope on the level above (Borg 1983)

![Figure 6](image2.png)

**Figure 6** Example of stress induced fracturing in the face of a Cut and Fill stope. The failure in the face is shown to indicate the requirements on the rock support in the back of the stope, as a failure zone of similar extent is developed also there
For Avoca mining and uppers on retreat, the numbers in figure 4 indicate the most critical areas with regard to high induced stresses and to difficulties of maintaining stability and a production free from disturbances. In both the Avoca mining and the retreat mining of sill pillars the brows (1 and 4) are the areas of the highest stresses and in these areas the rock is mostly failed. The backs of the stopes are often highly stressed in the Avoca mining (2) and often fail to some extent while the backs in the sill pillar mining (5) are mostly in a failed state. It is not unusual that the entire sill pillar is failed. In the Avoca mining the back of the upper drift (3) will be subject to increased stresses as the mining progresses. The area of influence in the drift is dependent on the integrity of the bench face, the larger the failure zone in the bench, the larger area of influence in the upper drift. In sill pillar mining, the remnants of the sill pillar (6) are left thick enough to maintain a support for the fill in the above lying stope.

However, the mining induced stresses do not only occur within the ore. In Cut and Fill mining the ore is reached from cross cuts driven from a local ramp for each ore body. As it is not economical to place the ramp at such large distance that it is outside the zone of mining induced stresses and deformations, the ramp will be affected by the change of stresses caused by the mining, especially for larger and less steeply dipping ore bodies. This can affect the rock conditions of the ramp and thus the requirements on the rock support, figure 7.

<table>
<thead>
<tr>
<th>Profil Y900</th>
<th>Stage 0</th>
<th>Stope mined</th>
<th>Sill pillars failed</th>
<th>Total extraction</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>J10-4 ramp</strong></td>
<td><img src="image1.png" alt="Image" /></td>
<td><img src="image2.png" alt="Image" /></td>
<td><img src="image3.png" alt="Image" /></td>
<td><img src="image4.png" alt="Image" /></td>
</tr>
<tr>
<td><strong>J-ramp</strong></td>
<td><img src="image5.png" alt="Image" /></td>
<td><img src="image6.png" alt="Image" /></td>
<td><img src="image7.png" alt="Image" /></td>
<td><img src="image8.png" alt="Image" /></td>
</tr>
</tbody>
</table>

Figure 7  The changes of differential stress around two local ramps as the ore in the vertical profile shown above is mined out (internal report).

Comparisons made between the stresses in numerical analysis and the development of failure around the stopes has shown that even small changes of the differential stress can cause fairly large increased damage.
of the rock and the rock support. This is most pronounced when the rock around the drift has already failed during the excavation and if the zone of the highest differential stress change rotates.

### 3.3 Room & pillar mining

Room and pillar mining is used in sub-horizontal ore bodies of relatively limited height. Within Boliden lead and zinc was mined in the Laisvall mine where 65 Mton of ore were mined from 1943 until the end of the mine 2001. The ore was located in a quartzitic sandstone of up to 30 m thickness and the mining left less than 20% of ore in the pillars as an average, see an example in figure 8. Pillar sizes varied between 6x6 to 10x10 meters and the widths of the drifts 10 to 15 meters depending on the location, the rock conditions and the height of the overburden. The strength of the pillars and its interaction with the surrounding mining in the more quartzitic areas were unexpectedly found out in the beginning of the seventies, figure 8 a) while the pillar strength for areas that included clay schists in the backs was investigated in a large scale pillar test (Krauland & Söder 1987).

Cut and Fill with pillars is used in the Kankberg mine where the ore widths does not allow full extraction using conventional Cut and Fill and in limited parts of the Tara mine where long hole stoping is not economically justified due to the limited height of the ore.

![Failed pillar in the Central zone](image1)

![Mining layout below Lake Storlaisan](image2)

Figure 8 a) Horizontal projection of an area below the Storlaisan Lake in the Laisvall mine. The pillars in the figure are 6x6 m and the drifts are 10 m, 60 m below the lake surface (20 m rock, 20 m sediments and 20 m water depth), b) failed pillar in the Central Zone

Normally Room & Pillar mining is used in fairly good rock conditions with relatively low requirements for rock support, as in the quarzitic sandstone in Laisvall. In Tara the need for rock support is governed by the need for back stability when mining towards layers of weak rock and in Kankberg by the occurrence of structures, figure 9, and to some extent also superficial spalling due to the stress concentrations in the backs in brittle rock conditions. The rock support is also adapted to the requirement that no rock falls are allowed to occur in any area where people are working.
In the Kankberg mine some areas are cut through by several subparallel structures resulting in blocky rock conditions and difficulties to maintain the integrity of pillars.

### 3.4 Long hole stoping

Long hole stoping is used in parts of the Garpenberg Mine, in Kylylahti and in most parts of the Tara Mine. Stopes are normally 10 to 15 m wide, 20 to 40 m long and about 30 m high, figure 10. Mining is done in primary/secondary fashion using stabilised backfill in the primaries. The Tara mine uses stabilized hydraulic fill, Kylylahti stabilized rock fill and Garpenberg paste fill for backfilling of the stopes.

![Figure 10](image.png)

a) Long hole stoping sequence at Garpenberg  

b) 3D-view of primary stopes at Tara

Figure 10  a) Long hole stoping sequence in Garpenberg in a vertical longitudinal profile looking towards the ore and b) long hole stoping in Tara shown as a 3D-view of primary stopes with one primary stope marked with blues lines.
As in the implementation of Cut and Fill mining, long hole stoping requires knowledge about the requirements of the properties of the fill, especially strength and how to fulfil that requirement. The implications of different mining sequences are very important, both with respect to the requirements on the fill properties and with regard to the induced stresses in secondary pillars, stope backs and in the sill pillar. The rock support of the stopes is designed to maintain stability of the ore not yet mined while the rock support of the development is designed for safety and stability. The development includes both drifts in the ore and the haulage drifts along the ore, including cross cuts towards the ore, figure 11, and local ramps. All of this development will be subject to mining induced stresses and deformations in a quite complicated manner as there will be both increase in stresses, with risk of failure depending on the stress level and the rock conditions, as destressing when the mining front has passed the levels. Figure 11 shows the results of an Examine 3D-model (Corkum, Curran & Grabinsky 1991) where the stresses in the pillars between the cross cuts were studied in detail. It showed a large risk of pillar failures but also decreasing stresses as the mining front had passed each level.

Figure 11  Stresses in the pillars between the cross cuts in long hole stoping in Garpenberg. As the mining front reach each level the stresses reach their maximum value and with continued mining the stresses decrease at that level (van Koppen 2008)

3.5  Open pit mining

Boliden is operating 3 open pits, Aitik and Maurliden mines in Sweden and the Kevitsa Mine in Finland.

Normally rock support is only used in smaller underground developments like pump stations and similar but meshing, shotcreting and rock bolting has been used in areas of poor rock conditions to eliminate the risk for rock falls. It has also been used when bench faces has been created along large scale structures with bench heights around 50 m with no space on the bench below for safety berms. Most of the work to maintain stability of the slopes is however based on knowledge of the structures and how they influence the stability of the benches together with adaption of the drilling and blasting technique to the rock conditions at different locations around the pit. The use of presplitting has been developed through the years and has been a key to increasing the slope angles at the foot wall of the Aitik pit (Marklund et. al 2007).
4 Principles of rock support

4.1 Planning of rock support

The planning of rock support is governed by requirements on safety and productivity, both internal requirements and those from the authorities. “Rock falls” is one of three prioritized risk for the Health & Safety of Boliden Mines. Eliminating the risk for rock falls implies controlling the rock conditions which also results in less disturbances of the production.

The process of planning rock support involves the following steps:
- compilation of available information
- risk analysis
- analysis of identified risks
- suggested measures to mitigate risks
- suggested procedures for follow up, e.g. monitoring

The level of detail and amount of work in each step depends on what level of investigation the risk analysis is performed. For the production planning the most important source of information is the follow up of the mining of previous or neighbouring stopes.

The actual design of the rock support in the Boliden mines is primarily based on experience. This is possible as experience is gained on a continuous basis from different kinds of excavations, mining conditions and rock support technologies. The choice of rock support can be further supported by general principles to support the experiences made. Historically, and currently, monitoring, follow up of the development of failures and both analytical and numerical analysis has helped to explain how and why the rock support works, or not, in specific conditions at the different mines.

In the production, rock support is planned for each stope and maps of the plans are used by the operators. The behaviour of the rock and the rock support is followed up through inspections and, to some extent, monitoring. The work follows the principles of the observational method (Peck 1969) where the observations made are compared to the expected behaviour and measures to mitigate potential problems are planned, or at least known, in advance.

4.2 Rock support elements

Table 1 shows a list of the rock support elements used in the Boliden mines.

Rock bolts are used to maintain and increase the strength of the rock itself through systematic bolting that allows bolts to interact, creating a beam of confined rock across and along the excavation, (Krauland 1983) and (Li, 2006 ). The distance between the bolts has shown to be more important than the length of the bolts in maintaining the rock closest to the surface intact (Rosengren et al 1992). In conditions where the rock fails between the bolts the bolting is complemented with surface support which in almost all cases consists of steel fibre reinforced shotcrete. A principal choice has been made to use denser rock bolt patterns rather than thicker shotcrete to control the deformations in the rocks closest to the excavation. A denser pattern of rock bolts decreases the depth of the rock not confined by the rock and reduces the load on the shotcrete, figure 12. The requirements of shotcrete thickness and on the load bearing capacity of bolt washer are also reduced.
Table 1 Types of rock support elements used in the Boliden mines.

<table>
<thead>
<tr>
<th>Type</th>
<th>Type, Standard pattern/thickness</th>
<th>Typical application</th>
<th>Special application</th>
</tr>
</thead>
<tbody>
<tr>
<td>Grouted rebar</td>
<td>Ø20 mm c/c 1-1.5 m</td>
<td>All areas</td>
<td>c/c 0.5-0.7 m, bolt/shotcrete arches</td>
</tr>
<tr>
<td>Fibre reinforced shotcrete</td>
<td>Steel fibres, 30-40 kg/m^3</td>
<td>All areas</td>
<td>100 mm, bolt/shotcrete arches</td>
</tr>
<tr>
<td>Bolt arches</td>
<td>c/c 5 – 10 m</td>
<td>Poor rock conditions, large deformations</td>
<td></td>
</tr>
<tr>
<td>Cable bolts</td>
<td>1-2x15.2 mm, c/c 2-2.3 m</td>
<td>long hole stopes</td>
<td>large cross cuts</td>
</tr>
<tr>
<td>mesh, static</td>
<td>chainlink mesh</td>
<td>Infrastructure in poor rock conditions</td>
<td></td>
</tr>
<tr>
<td>mesh, dynamic</td>
<td>welded mesh</td>
<td>Ø6 mm x 75 mm* square</td>
<td>dynamic rock support</td>
</tr>
<tr>
<td>D-bolt</td>
<td>Ø 22 mm, 2.0-2.7 m*</td>
<td>c/c 1.5 m*</td>
<td>dynamic rock support</td>
</tr>
</tbody>
</table>

* The dynamic rock support has so far only been installed in previously supported areas, on top of the standard rock support. Areas has been identified where there is a need for dynamic rock support but has not yet been mined.

Figure 12 The tensile stresses in the bolts create a zone of confined rock, marked with squares in the figure. The unconfined rock closest to the surface between the bolts is supported by the shotcrete layer. Modified after (Li 2006).

Grouted rebar is used almost extensively. Even though it does not fulfil all requirements in all locations it is still chosen as the main alternative as it cannot be motivated to use more expensive bolts in general. The cost to install more rebar where it is required is lower than to install a better bolt in general. In very poor rock conditions and for dynamic rock support the D-bolt is the preferred alternative due to its ability to sustain large deformations, both in tensile and shear loading (Chen & Li 2015).

To further increase the strength of the rock in very poor mining conditions “bolt arches” are used. A bolt arch consists of an extra row of bolts that is placed in between two ordinary rows of bolts at a c/c-distance of 0.5-0.7 m and combined with fibre reinforced shotcrete, (Li 2006; Krauland, Board and Marklund 2001). This increases significantly the installed amount of rock support at that location and increases the load bearing capacity of the rock itself. The distance between these arches is usually 5-10 m depending on the situation. It is also often used as support for the brows in Avoca mining and sill pillar mining, figure 4, but
also for brows in the cross cuts into long hole stopes. Meshing is used only to a very limited extent in the Boliden mines.

Cable bolting is mainly used in long hole mining. It is used to support the backs in weak layers of rock in the hanging wall at Tara and in secondary stopes in Garpenberg to maintain the integrity of the ore in the back as they are both excavated to the full width.

Important factors for the required rock support are the use of perimeter blasting and mechanised scaling. The perimeter blasting reduces the damage zone and allows for a smooth profile which is further enhanced by the mechanised scaling. The mechanised scaling also more or less eliminates the risk for larger amounts of loose rock to be left until shotcreting and bolting, thus reducing the risks of larger rock falls that could impose risks for both operators and equipment.

Another parameter that is used to control the amount of failed rock in each round is round length. The standard boom on the production drill rigs is 5.4 m which results in about 5 m round length. By decreasing the round length the failure zone and the risk for rock falls decrease significantly. A “short” round is normally 3.5-4 m.

5 Rock support case studies

In the following a number of case studies will be presented that show some different situations that has formed the basis to the descriptions earlier in this paper.

5.1 Cut and fill

Figure 13 shows the principles of failures in a Cut and Fill subjected to high stresses n poor rock conditions, and figure 14 a) a picture from a more recent stope. The profile is uneven due to failure and fall outs after blasting and during scaling. Large deformations indicated by roof levelling indicated the need for increased rock support and bolt arches were installed. In the figure it can be seen that the profile of the last round is more or less as planned with a horizontal back and no fall outs. It is the result of the use of shorter rounds in order to decrease the failure zone directly after blasting.

![Figure 13 Failure principles for a Cut and Fill stope in the beginning of the 90's (Board et al 1992)](image-url)
Figure 14  a) Bolt and shotcrete arch in a Cut and Fill stope. Chlorite alterations in the contact zones and in the ore combined with high induced stresses cause difficulties to maintain a smooth profile. b) shows typical failure modes around the perimeter of the stope: 1) circular failure in the foot wall, 2) shear failure and movements along the ore contacts, 3a) tensile failures along the contacts due to high induced horizontal stresses (3b) around the stope, 4) bending and tensile failures in the hanging wall, 5) failed zone around the stope (grey area , often outside the bolt length).

Figure 15 a) shows another example of the use of bolt arches. The figure shows the upper drift, marked 1 in figure 15 b), in an Avoca mining. As the upper drift is mined the bench/pillar (2) between the mucking level (3) and the upper level fails and the stresses in the pillar and in the back of the upper level increase. This often results in failure of the rock in the back and in the walls of the lower drift (3) but in some cases also in the upper drift. In figure 15 a) the effect of the bolt arches is clear as it decrease the amount of deformation and more or less eliminates the risk for fall outs. Between the bolt arches the rock is allowed to deform and in some cases rehabilitation is required, as in the figure where mechanised scaling has been used to clean the rock from failed shotcrete due to too large deformations.
a) Upper drift, marked “1” in the profile in b)  
b) Vertical profile and vertical longitudinal profile along an Avoca mining stope

Figure 15  Bolt and shotcrete arch in the lower drift in Avoca mining in Garpenberg. The arch supports the rock which strength increases through the resulting confinement. Between the arches the deformations are larger and in this case scaling has had to be performed.

5.2 Room & pillar mining

During mining of the cross cuts in the pillars in the Laisan 3 area, figure 8, as many as 40-50 pillars failed in something that seemed like a shear deformation between the back and the roof, Figure 16. Roof levelling showed a time dependent convergence. The area was backfilled with hydraulic back fill (tailings from the concentrator) and the deformations stopped, most probably by the increased strength the pillars gained through the confinement from the pressure of the back fill (Knutsson 1983). This is a very good example of how rock support does not always have to be made up of steel and concrete.

Figure 16  Pillar failed in shear between back and floor in the Laisan 2 area from Figure 8, the right picture showing the contour of the pillar and the location of the crack caused by the shearing. The rock in the pillars failed between horizontal weakness planes
5.3 Long hole stoping

Figure 17 shows a pillar between cross cuts in the long hole stoping in Garpenberg, see also figure 11. The pillar ends had failed due to the high stresses resulting from the mining but was supported by bolting and shotcrete. When seismic events in the range of $M_R 1-1.5$ occurred the support could not hold the loose rock and fairly large rock falls occurred from 3 pillars in the area. In the haulage drift, parallel to the ore contact, there was significant heaving of the floor, also as a result of the seismicity. As there had not been any seismicity in this area before, the rock support had to be adapted to the new requirements that now had become obvious. The standard rock support has been complemented with mesh and D-bolts to sustain the rock and to contain the damage resulting from similar events, should they occur in the area (Figure 18).

Cable bolting is used to stabilise the secondary stopes as the ore is suspended on the paste fill of the backfilled primary stopes, figure 19.

Figure 17 Pillar between cross cuts that was failed due to high stresses and then was subjected to a seismic event. The picture is taken towards the ore, into the cross cut.

Figure 18 The dynamic rock support where the rebar and shotcrete has been complemented with mesh and D-bolts.
Figure 19 Vertical profile towards the stopes in long hole stoping in the Garpenberg Mine. The 15 m wide secondary stopes are cable bolted using 15.2 mm cables.

5.4 Open pit mining

Rock support has been used in a few locations with very specific rock conditions and/or geometries. In the large open pit in Aitik there are a couple of locations where meshing has been used to eliminate the risk for rock falls where crushed zones cut through benches above locations where people work. In Maurliden a part of the pit wall was designed to follow a large scale structure, leading to a 50 m high bench face with no space for safety berms below. Mesh and short bolts was installed to eliminate the risk for rock falls, figure 20.

Figure 20 A part of the pit walls of the Maurliden open pit where mesh is used to eliminate the risk for rock falls. The figure shows the mesh during installation.
In the Aitik open pit the major effort is made to mitigate the risk for rock falls and to fulfil the requirements of the bench widths by adapting the drilling and blasting to the requirements for the different design sectors of the pit. On the foot wall this implies the use of presplitting in holes dipping 70 degrees from the horizontal line (Marklund et al 2007).

![Figure 21 The result of presplitting of the foot wall of the Aitik open pit](image)

## 6 Productivity

With the relative large amount of rock support installed in each round and the fact that it includes three operations, scaling, shotcreting, and rock bolting, the productivity of the installations becomes an important factor for the total productivity. The total productivity of the mining includes factors such as planning, number of faces, cycle times, etc. that are not discussed here.

The scaling is done mechanically with hydraulic hammers. The time to scale a round is governed by the rock conditions. To cover all surfaces of a round, good rock conditions takes less than an hour. With increasing amount of loose or failed rock or geological disturbances the time increases. Different techniques, i.e. water scaling, have been tested but did not satisfy the requirements. It has been difficult to increase the productivity of the scaling itself, but the need for scaling can be reduced by the use of shorter rounds and controlled perimeter blasting.

The productivity in the shotcreting operation has been studied very little. The reason is that the time for the actual spraying of shotcrete is short; typically 45 minutes for spraying 6 – 9 m³ and covering roof and walls of one round. The transport of concrete to the spraying unit is often the limiting factor, not the spraying itself. The curing time required before the bolting is allowed can be reduced through the use of accelerators but the requirement of a minimum strength in poor rock conditions still requires a specified curing time for the shotcrete.

Rock bolting is thus the operation where productivity is most important and can be affected. Studies of this in Boliden Mines have been made under the MIGS* program and also in a thesis work (Karlsson & Taavoniku 2015).

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* MIGS, The Mining Initiative on Ground Support and equipment program is a project run by RTC, The Nordic Rock Tech Centre, focusing on examining developmental and innovative opportunities in the areas of ground support and equipment. It was initiated 2007 and the third MIGS-program was started 2016.
In the actual bolting cycle (drill hole – place cement/resin – insert bolt – reload bolts – replace bits), the capacity is in the order of 10–11 bolts per hour. In the toughest rocks it is down to 7–8 bolts per hour. Adding operators, repair, cleaning, transportation between faces and lunch break, the “operational capacity” drops to 5–7 bolts per hour.

There are small differences in productivity between machines using the different grouting methods and between the larger manufacturers. The cement machines have higher cycle capacity but due to more time on cleaning, the long term capacity is equal as that for resin grouting.

The next step to increase productivity will probably come from automation, where the drilling and setting of bolts is automated and the machine is “fed” with new bolts and grouting material.

7 Concluding remarks

The case studies presented above, and experience from other mines, show that there are methods and technologies available that in combination will solve most rock support problems experienced in a large variety of rock conditions and mining methods. This is valid especially if we include the mining technique, e.g. perimeter blasting and adaption of the exposed, unsupported areas to the rock conditions (e.g. round length while drifting). The challenge is to know when and where to use the available solutions. It requires not only knowledge of the behaviour of different rock masses in different situations but also what rock conditions are to be expected, in other words geological and geotechnical information. There is also a design problem as it is not always the most economical solution to base all design on experience. The rock support should be adapted to the specific conditions at each location and even though there are many similarities between different locations in a mine it is not always the same situation. The design problem is especially evident for dynamic support where little is known about the relationship between a seismic event and the actual load on the rock support.

The development of mining equipment and of the rock mechanical understanding is more or less an ongoing process where small steps are taken all the time. The technological development in monitoring together with the introduction of Wi-Fi in the mines has in relatively short time opened up new possibilities for monitoring and for gathering of information. Drones, bolts that are individuals in a network reporting its status and scanning are all very interesting techniques that will increase the possibilities for monitoring of stability but the large amount of information will also allow for more detailed analysis for increased knowledge on how the rock and the rock support reacts to the applied loads. Seismic tomography based on the background noise of a seismic monitoring system is being developed as a tool to monitor stress change and will increase the understanding of the rock mass behaviour around mine openings.

Boliden is involved in several development projects in the fields mentioned above but also in projects aiming at increased productivity of the equipment and the planning processes for more efficient planning and installation of rock support.

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Tunnelling methodology for difficult rock mass conditions in deep subsea tunnels

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Abstract

Tunnelling deep below the sea floor involves great challenges related to excavation and rock support. In this paper such challenges are discussed based on experience from Norwegian hard rock subsea tunnels. The most challenging rock mass conditions for these tunnels have been caused by major faults/weakness zones. Poor stability weakness zones with large water inflow have been particularly difficult. At the pre-construction investigation stage, geological and engineering geological mapping, refraction seismic investigation and core drilling are the most important methods for identifying potentially adverse rock mass conditions. During excavation, continuous engineering geological mapping and probe drilling ahead of the face are carried out, and for the most recent Norwegian subsea tunnel projects, MWD (Measurement While Drilling) has been used. During excavation, grouting ahead of the tunnel face is carried out whenever required according to the results from probe drilling. Sealing of water inflow by pre-grouting is particularly important before tunnelling into a section of poor rock mass quality. When excavating through weakness zones, a special methodology based on spiling bolts, short blast round lengths and installation of reinforced sprayed concrete arches close to the face is normally applied. The basic aspects of investigation, excavation and support when crossing major weakness zones are discussed in this paper and illustrated by cases representing two very challenging projects which were recently completed (Atlantic Ocean tunnel and T-connection), one which is under construction (Ryfast) and one which is planned to be built in the near future (Rogfast).

1 Introduction

Since the early 1980’s around 50 subsea rock tunnels have been built along the coast of Norway (NFF 2009, Nilsen & Henning 2009). Most of these are road tunnels, with the 7.9 km long Bømlofjord tunnel as currently the longest, and the Eiksund tunnel as the deepest, with its lowest section 287 m below sea level. Some subsea tunnels have also been built for the oil industry as shore approaches and pipeline tunnels, and some have been built for water supply and sewerage.

Extensive site investigations, based on offshore acoustical profiling, refraction seismics and in most cases also core drilling in addition to conventional onshore mapping, are always carried out for the subsea tunnels. In addition, extensive investigations are carried out during excavation. In many cases, excavation of the Norwegian subsea tunnels has been completed without major problems related to the ground conditions. In difficult ground conditions, tunnelling challenges have in most cases been efficiently tackled by thorough investigation from the tunnel face and well adapted procedures for pre-grouting, excavation and rock support. The most difficult rock mass conditions have been represented by major faults/weakness zones with large water inflow.

The initial part of this paper will discuss the challenges related to predicting zones of adverse rock mass conditions prior to excavation in order to be able to choose optimum tunnelling methodology for excavation through such conditions. After that, main focus will be on discussing the principles for excavation and rock support in such conditions. For illustration, two relevant, recently completed projects (Atlantic Ocean tunnel and the T-connection) will be discussed in some detail, and two very long and deep subsea tunnels under construction and in planning (Ryfast and Rogfast, respectively) will be briefly
described. The paper is based mainly on the author’s experience as member of expert panels for many Norwegian subsea projects.

2 Ground investigations

2.1 Prior to excavation

The main pre-construction investigations for a subsea tunnel are:

1. Desk study
2. Onshore engineering geological mapping
3. Reflection seismics
4. Refraction seismics
5. Core drilling

The desk study includes review of geological maps, reports, aerial photos and experience from any nearby projects, and represents the important first step of the investigations as well as a basis for the planning of further investigation of the project area.

The onshore mapping includes conventional geological mapping to determine rock types, detail jointing, major geological structures such as faults, dikes, lithological contacts, and most importantly; major weakness zones in the planned tunnel area.

Reflection seismic investigation (often referred to as acoustic profiling) is used for finding the depths to different geological layers (reflectors), including the depth to the bedrock surface where it is covered by loose deposits. Refraction seismic results are used for “calibration” of estimated sonic velocities. The bedrock for some of the Norwegian subsea tunnels is located below as much as 200m of sediments. The main target for this type of survey is to get an overall view of the soil distribution in the area to produce a map of the rock surface. These maps based on reflection seismics are of great importance for identifying favourable corridors for subsea tunnel crossing.

Refraction seismic investigation is performed by positioning a cable with hydrophones on the sea bottom and detonating small charges of dynamite. Based on monitoring the arrival time of the refracted waves the thickness of soil cover and sections of different sonic velocities, particularly low-velocity sections representing potential weakness zones, are identified as illustrated in Figure 1. Interpretation of seismic velocities and thickness of the various layers is a complex process, and a great deal of operational experience is required for the results presented in a profile to be reliable.

Seismic velocities higher than 5,000 m/s generally indicate good quality rock masses, while the poor quality rock mass of weakness zones have velocities lower than 4,000 m/s. In some cases seismic velocities lower than 2,500 m/s, corresponding to the velocity of moraine, have been monitored for weakness zones.

Core drilling is used to obtain information from deeper layers of the rock mass and is often used in combination with geophysical measurement as shown in Figure 1. Based on core drilling, more detailed information on rock mass structures can be collected, such as degree of fracturing (RQD), character and orientation of weakness zones, and further information on the rock types and their boundaries in the rock mass can be obtained. Also, samples can be collected for laboratory testing and petrographic analyses, and the drill hole can be used for testing and investigation, i.e. permeability testing (Lugeon test).

For the subsea tunnels, core drilling has in most cases been carried out from the shore as illustrated in Figure 1, but in some cases it has been carried out as directional drilling as shown in Figure 2. In this case directional drilling of a 900 m long hole (BH-1) made it possible to detect a deep erosion channel in time sufficient for adjusting the planned alignment. By lowering the alignment 30 m, large stability problems were avoided, and considerable time and money were saved. In a few cases, when this has been
considered necessary to prove the feasibility of the project, core drilling has been carried out from drill ships.

Figure 1  Use of seismic investigation and core drilling for planning of subsea tunnel. Dotted lines represent interpreted rock surface based on reflection seismics. Interpreted velocities based on refraction seismics are 3,500-5,500 m/s in rock and about 1,700 m/s in soil. RQD and Lugeon-values (L) are indicated by histograms along the core drill hole.

Figure 2  Directional drilling of more than 900 m long core drill hole at the Bømlafjord subsea tunnel (from Palmstrøm et al. 2003)

2.2  Investigations during excavation

Even the most extensive pre-construction investigations cannot reveal all details regarding rock conditions. Some uncertainty will always remain when tunnelling starts. To avoid any “unexpected conditions”, and at all times have good control, systematic probe drilling during tunnelling is very important. Normally, probing is done as percussive drilling by using the tunnel jumbo, and 3-5 holes are used under water according to procedures as shown in Figure 3. As can be seen from the Figure, probe drilling also has the very important purpose of providing the basis for deciding whether to grout or not. This will be further discussed in the next section of this paper.

Figure 3  Principles of probe drilling (stippled lines) and pre-grouting (solid lines). Typical length of probe drilling holes is 25-30 m, and overlap with the previous probe drill round is typically about 5 m (from Nilsen & Palmstrøm, 2013)
The most difficult rock mass conditions often occur in the fault zones at the deepest part of the tunnel. Any uncontrolled major water inflow here may have severe consequences. In such sections of the tunnel, core drilling is sometimes used for probe drilling.

In addition to probe drilling, continuous follow-up at the tunnel face by experienced engineering geologists and rock engineers is of great importance. In Norwegian tunnelling this has become more and more realized, and time for such follow up is today included in the contract (as a special item in the contract called “the owner’s half hour”).

For the more recent projects, MWD (Measurement While Drilling) and DPI (Drill Parameter Interpretation) have been applied for predicting rock mass conditions ahead of the tunnel face, and for future projects this will now be standard procedure. Three main factors describing the rock mass conditions are normally defined by this approach; rock hardness (strength), degree of fracturing and water conditions. The potential of MWD/DPI for estimating conditions ahead of the face is illustrated by Figure 4.

Use of MWD/DPI has a great potential for predicting rock mass conditions ahead of the tunnel face. The method is however still at the development stage, and interpretation of data has in some cases been slow and uncertain. As basis for the decision on whether to pre-grout or not, measurement of water inflow in probe drill holes is therefore still the preferred method.

3 Methodology for tunnelling through difficult rock mass conditions

3.1 Excavation

All Norwegian subsea tunnels so far have been excavated by drilling and blasting, which provides great flexibility for varying rock mass conditions and is cost effective. The 6.8 km North Cape tunnel (completed in 1999) was considered for TBM, but also in this case drilling and blasting (D&B) was chosen as the final method. A main reason for not choosing TBM was that the risks connected to potential water inflow were considered too high. During tunnelling, water inflow was however not a main problem. The main problem
turned out to be thinly bedded rock causing stability problems in the D&B drives, which due to the uniform circular profile and less disturbance of the contour by TBM-excavation probably would have been less in a TBM drive.

Water sealing by pre-grouting is carried out when required according to criteria based on probe drilling. For Norwegian subsea road tunnels today a maximum inflow of 3 l/min for one probe drill hole and a total of 10 l/min for 4 holes are typical action values for pre-grouting. By applying such criteria, the remaining inflow can be controlled and adapted to preset quantities for economical pumping (normally a maximum of 300 litres/min-km).

Grouting, when required according to probe drilling, is always carried out as pre-grouting in drillholes typically about 25 m ahead of the face (see Figure 3), and with 2 blast rounds overlap. This procedure has been successful even in the deepest of the Norwegian subsea tunnels where grouting against water pressures of 2-3 MPa has been efficiently performed with modern packers, pumps and grouting materials. Grouting pressures up to 10 MPa are today quite common with modern grouting rigs as shown in Figure 5.

![Modern equipment for high pressure pre-grouting](image)

In one extreme case, the Oslofjord subsea road tunnel (completed in 2000), where a moraine-filled depression was encountered during tunnelling (similar to the situation in Figure 2), ground freezing was required for safe excavation through the difficult section. Although preparedness for this option is still often included in the contract, this is considered a realistic option only for very special cases like the one that was encountered at the Oslofjord tunnel.

### 3.2 Rock support and lining

As rock support in the subsea tunnels, a combination of fibre reinforced shotcrete and rock bolting is most commonly used. The shotcrete is most commonly applied as minimum 8 cm thick, wet mix, polypropylene (PP) fibre reinforced. The rock bolts have extensive corrosion protection. The preferred bolt type is one providing multiple corrosion protection by hot-dip galvanizing, epoxy coating and cement grouting applied on both sides of a plastic sleeve. In good quality rock, spot bolting is sometimes sufficient, while in poorer quality systematic bolting is common.

In difficult ground conditions spiling bolts are used, often combined with reinforced shotcrete ribs as illustrated in Figure 6. When the conditions are particularly challenging, reduced round length (down to 1-2 m instead of the conventional 5 used in good rock) and stepwise excavation of the face are applied. The
trend today is that shotcrete ribs (sometimes supplemented with concrete invert) are used in poor rock conditions instead of concrete lining.

In very poor quality rock mass, cast-in-place concrete is however still in some cases required as final lining, and in some such cases the lining is installed close to the face as illustrated in Figure 7. In a few extreme cases, mainly when swelling clay has been the main challenge, it has been necessary to install lining also in the invert.

All rock support structures are drained, whether they are made of cast-in-place concrete lining, shotcrete ribs or shotcrete/rock bolting, and during operation leakage water is continuously pumped out of the tunnel. The basic principle for stability support (in good/fair quality rock) and final lining for preventing water from dripping on the tunnel floor is shown in Figure 8.
Figure 8  Principle sketch of inner water/frost lining commonly used in Norwegian road tunnels

4Case examples

4.1 Recently completed tunnels

To illustrate the very challenging rock mass conditions that have in some cases been encountered in Norwegian subsea tunnelling, and the way the problems have been solved, two relevant, recent cases will be briefly discussed; the T-connection and the Atlantic Ocean tunnel.

4.1.1 The T-connection

The “T-connection” represents a part of the main coastal road between Haugesund and Stavanger on the SW coast of Norway (see Figure 13). The main tunnel (Karm sund tunnel) is 7.2 km long with 70 m² cross sectional area (profile T9.5), and in addition the project includes a 1.2 km long branch tunnel from an underground roundabout, see Figure 9. The deepest points below the two fjords are 139 m and 136 m, respectively. The tunnels were excavated in 2009-2011, and the project opened for traffic in 2013.

Because of the very difficult ground conditions encountered in the Statpipe tunnels, and since no core drilling was carried out at the pre-construction stage for the T-connection, exploratory drilling ahead of the tunnel face was performed almost for the entire tunnel length. No significant water inflows were encountered, and the extent of pre-grouting therefore was moderate.
As shown by Figure 9, the T-connection tunnels were excavated in greenstone/greenshist, sandstone, phyllite and gneiss. The degree of jointing was mainly moderate. There were, however, many small weakness zones (fault and shears) and a few large. Still, the T-connection tunnels did not encounter quite as problematic rocks as the nearby gas pipeline tunnel.

Two large weakness zones (thickest steep lines in the profile in Figure 9) represented the most challenging tunnelling conditions. Here, the blast round length was reduced from 5 to 3.5 m, and 6-8 m long spiling bolts with 3 m overlap were installed in roof and walls before blasting, see Figure 10. Thick fibre reinforced shotcrete with rebar reinforced arches and rock bolts were used for temporary and permanent support.

The Atlantic Ocean tunnel, located on the central west coast of Norway, is a 5.7 km long subsea tunnel with excavated cross section of approx. 85 m². A longitudinal profile of the tunnel is shown in Figure 11. The tunnel was opened for traffic in 2009.

The bedrock is Precambrian granitic gneiss of mainly good quality. The conventional pre-construction investigations for this type of project were carried out, including reflection and refraction seismic investigations. Based on the latter, several low velocity zones, representing faults/weakness zones under water were detected. Near the bottom of the planned tunnel zones with seismic velocities as low as 2,500 and 2,800 m/s were identified as shown in Figure 11. Based on overall evaluation of the conditions, a
minimum rock cover of 45 m was accepted, but it was realized that several of the low velocity zones under sea could be challenging, and this was taken into account in the planning of excavation and rock support.

From the west, before entering a major zone at Station 6242, several nearby fault zones with seismic velocity down to 2.8-3.1 km/s, and even down to 2.4 km/s from the east side, had been crossed without major problems. These zones contained crushed rock and clay gouge, but very little water. Probe drilling indicated poor quality rock in the 2.8 km/s zone at St. 6242, but little water inflow. Thus, similar rock mass conditions as in the previous faults/weakness zones were expected. As extra precaution, the great water depth and limited rock cover taken into consideration, grouting was carried out in order to seal joints and possibly also stabilize the zone material. After that excavation was started with reduced round length (3 m), shotcreting, systematic radial bolting and installation of 6 m long spiling bolts.

The weakness zone proved to be of very poor quality, and after blasting the reduced round length there was a tendency of small rock fragments to fall down between spiling bolts. Attempts to stop this by applying shotcrete were unsuccessful, and after a few hours a 5-6 m high cave-in of the roof had developed, covering the full tunnel width and the 3 m round length. Based on holes drilled later it was found likely that the cave in progressed about 10 m above the tunnel roof.

In order to stabilize the tunnel, excavated material had to be filled up against the tunnel face and a more than 10 m long concrete plug was established to seal the tunnel. Probe drilling indicated considerable water leakage, and extensive grouting of the backfill material and the surrounding rock past the slide scar was required. After having established the concrete plug, based on careful excavation with reduced round lengths, shotcreting/radial bolting and spiling with drillable rock bolts, the tunnel face was re-established after 5.5 weeks at the same position as it was before the cave-in. Core drilling through the weakness zone showed that it was more than 25 m wide and had considerable water leakage.

Further tunnelling was carried out based on a procedure with extensive pre-grouting, spiling, excavation with reduced round lengths/piece by piece, shotcreting/radial bolting and installation of reinforced ribs of shotcrete. Support of the tunnel face as illustrated in Figure 12 was also required. The process was very time consuming due to extensive water leakages (up to 500 l/min in one single drill hole) at very high pressure (up to 23 bar). Tunnelling was continued approx. 20 m from the west side, and this position was reached about 10 months after the date of the cave in. The rest of the fault zone was excavated from the east side based on a similar procedure as described above. As permanent support in the central zone a full concrete lining, including heavily reinforced invert lining, was established (see Figure 7).

More than 1000 tons of grout (mainly micro cement, but also standard cement and polyurethane) was needed to seal the leakages of the approximately 25 m wide fault/weakness zone. After completion of the
Tunnelling methodology for difficult rock mass conditions in deep subsea tunnels

B. Nilsen

4.2 Projects under construction/in planning

Several new, very long and deep subsea tunnel projects will be built in the near future, including the Ryfast (currently under construction) and Rogfast projects. These are located only about 30 and 20 km, respectively, south of the T-connection project, see Figure 13.

Figure 13 Locations of the T-connection project (completed), Ryfast (under construction) and Rogfast (in planning)

Ryfast includes two tunnels: the Solbakk tunnel and the Hundvåg tunnel, both with two tubes 12 m apart. Each tube has a span of 9 m (70 m$^2$ cross section) and cross passages for every 250 m. The Solbakk tunnel will be 14 km long when completed, and descends down to -290 m below sea level. It is passing through various gneisses, and several large weakness zones are expected. The Hundvåg tunnel will be 5.5 km long when completed, with phyllites at the southern part, and gneiss in the rest. Construction of Ryfast started in 2013 and planned opening of the link is in 2019.

The Ryfast tunnels are being excavated by drill and blast. Difficult rock mass are to be expected for sections of the tunnel. It is estimated that 250,000 rock bolts and 100,000 m$^3$ of shotcrete will be used for rock support, plus cast in place concrete lining in very poor ground conditions. So far, tunnelling through weakness zones has been performed without any big problems. The cost for the project is estimated at 5,500 mill. NOK.

Rogfast, which is now at its final investigation and planning stage, is also planned with two separate tubes, each with two lanes. Each tube will be about 26.7 km long and go down to a deepest level of about 385 m below sea level. The project is planned with connection approximately midway to the island Kvitsøy. The structural geology of the project area is very complex, with several major faults and thrust zones, and with phyllite as predominant rock type in south, gabbro and greenstone in the middle and gneiss in north. Ground investigation is particularly challenging because of the long sections under open, deep sea.

The conditions are expected to be very challenging for Rogfast, with several poor quality weakness zones as illustrated by the drill cores in Figure 14. Extensive investigations have been performed, including
directional core drilling from drill ships at sea depths of up to 290 m. The cost of Rogfast is estimated at 10,200 mill. NOK, and earliest start of construction is estimated to 2017.

Figure 14 Example of poor quality rock mass from core drilling at Rogfast (black, thin sections are tubes representing core loss)

5 Concluding remarks

This review of Norwegian projects illustrates that during excavation of subsea tunnels, even in hard rock, very challenging conditions are often encountered. The most difficult conditions are represented by major faults and weakness zones, particularly when very poor rock mass quality is combined with high water inflow. Even in such cases, the Norwegian projects have demonstrated that with the technologies regarding pre-grouting, tunnelling and rock support which are available today, such challenges may be successfully coped with. If properly planned, designed and performed, a leaner, faster and more cost-effective alternative to full concrete lining, consisting of ribs of reinforced shotcrete, can in many cases be used as permanent support.

For any subsea tunnel project extensive, well planned and professionally performed pre-construction investigations, continuous investigations during tunnelling, appropriate procedures for excavation/rock support and high state of readiness are crucial. This applies even more for very challenging subsea tunnel projects like Rogfast, which is planned to be built in the near future on the southwest coast of Norway. The long experience from the many completed subsea tunnels in Norway, and particularly the lessons learned from projects such as the Atlantic Ocean tunnel, the T-connection and others, undoubtedly will have a great value for the planning and safe completion of this project.

References

Rock support in the Malmberget mine

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Abstract

The Malmberget Mine is an iron ore mine operated by LKAB (Loussavaara-Kiirunavaara AB) located close to the town of Malmberget. The mine consists of around 20 orebodies of varying sizes and shapes and has been mined since 1888. The mining method is large scale sublevel caving which is cost-effective and allows for a high degree of mechanization and automation. The main disadvantages are subsidence of ground surface and, for the city of Malmberget, vibrations caused by seismic events mainly due to the caving process of hanging wall areas. Mining induced seismicity is also causing rock falls underground and to prevent falls of ground in the Malmberget mine a rock support system designed against mining induced seismicity is used.

This paper describes the rock support system used by the Malmberget mine. It also describes how a concrete tunnel was used to make a passage through a collapsed section of a strategically important drift.

1 Introduction

The Malmberget mine is an iron ore mine owned by LKAB (Loussavaara-Kiirunavaara AB) located close to the city of Malmberget. Malmberget city (Figure 1) is located in northern Sweden, approximately 70 kilometres north of the Arctic Circle.

Figure 1  Aerial view of Malmberget
The presence of ore has been known for centuries; originally the ore was transported from the mine by horses and reindeer. In 1888, the railway to the coastal city of Luleå was completed, and large scale production began. Until today most of the refined iron ore products from Malmberget, such as fines and pellets, are transported to Luleå for use at steel mills mainly within the Baltic Sea region. The mine consists of around 20 orebodies of varying sizes and shapes, see Figure 2.

![Figure 2](image1.png)

**Figure 2** Orebodies in Malmberget, metric scale, plan view

Production in 2015 was 16 million metric tons of crude ore from 13 of the orebodies; the deepest production was approximately 900 metres below ground surface. The main mineral is magnetite but there are also smaller quantities of hematite. The mine is operated by LKAB. The city of Malmberget has existed in close relationship with the mine for well over a hundred years. The first houses were built by miners who wanted to live close to their place of work. More miners followed and soon a shanty town, resembling those found in the Klondike during the gold rush era, was established on the hill slopes of Malmberget (Figure 3).

![Figure 3](image2.png)

**Figure 3** Malmberget main street 1895, from LKAB archives
The rich ore deposits were not depleted and as the town grew the old shacks were eventually replaced by modern buildings. Mining started in open pits, but by the 1920’s more than 90% of all mining was underground. Many orebodies dip to the southwest which gradually brings mining activities closer to the city itself. Since the 1960s sublevel caving has been the predominant mining method. It is a large scale method where different activities take place on several levels simultaneously (Figure 4).

Figure 4 Sublevel caving

This mining method makes it possible to efficiently mine deposits at large depth at competitive cost. The main disadvantages are subsidence of ground surface and, for Malmberget city, vibrations caused by seismic events mainly due to the caving process of hanging wall areas (Figure 5).

Figure 5 Environmental impact
Buildings, residential areas and infrastructure have been relocated due to the local presence of mining for more than 50 years. This process is generally well accepted among the residents of Malmberget. Inconveniences caused by seismic events is, however, a relatively new problem. In order to ensure the continuation of mining in Malmberget, an extensive, city relocation program has begun.

As mining progresses deeper, the magnitude of the rock stresses increases. Due to the large scale of the mining method, stress concentrations emerge underneath the orebody, and peaks underneath the production level (Figure 6). The demand on the rock support system has changed over time as the mining induced seismicity has become a reality, and since 2010, the standard rock support is designed to withstand seismically induced dynamic load.

![Figure 6](image_url)  
**Figure 6** Schematic stress patterns of the mining induced rock stress state in a large sublevel caving mine

## 2 Rock support in the Malmberget mine

LKAB uses standards for rock support at the Malmberget Mine. This is to increase the safety as well as to provide an opportunity for validation of the function of the rock support system. The standards include rock support classes which are based on geology and different load situations. In short, the rock support system can be divided into a design for static load and a design for dynamic load, i.e. load from mining induced seismicity. The exception is in areas with massive biotite schists, where large deformations are anticipated. In these areas the design is similar to the system designed for dynamic loads. The rock support system for static load conditions consist of steel-fiber reinforced shotcrete and rock bolts (Figure 7). In 4-way intersections and other areas where the span is greater than 9 m, the bolts in the roof are replaced with cable bolts (at least 7 m long).
In areas where there is a risk of seismic events, a rock support system for dynamic load conditions is used. It consists of steel-fiber reinforced shotcrete, steel mesh (outside the shotcrete) and dynamic rock bolts (D-bolt) (Figure 8).

Since the dynamic rock support only has been used since 2010 and was only small-scale in the beginning, there is no data available to measure how effective it has been. Reporting of accidents and incidents due to falling rock started in 2000 and it is believed that by 2005 most of the incidents were routinely reported, which was not the case from the start. Since 2009, shotcrete is applied on the walls and the roof of the drift after every blast and it can be seen that the number of incidents has decreased since the beginning of this practice (Figure 9). This implies that most of the reported incidents are from smaller fallouts when working close to the face.

There has been an increase of incidents due to falling rock since 2013. This is probably the result of an increase in both the number and magnitude of the seismic events in Malmberget.
Sometimes the standards for rock support are not enough and extraordinary measures have to be used to ensure stability of the drifts. The concrete tunnel on the M1250 level is such a case.

The new main transportation, level M1250 in the Malmberget Mine, was officially taken into operation during 2011, at 1250 meters depth below surface. All iron ore loading occurs above the transportation level, and the ore flows to the transportation level through orepasses. On M1250, the ore is emptied from the orepasses to diesel trucks that carry up to 90 tonnes of ore from the 12 different orepasses to the central crushers. After crushing, the ore is hoisted to the processing plants on the surface for further handling. The length of the transportation routes for the diesel trucks are considerable since the M1250 level is spread over an area of approximately 2.3 x 1.6 kilometers, which makes it important to maintain an efficient flow of ore.

On August 28th 2009, a transportation drift to orepass 225 was being developed when the roof of the drift suddenly collapsed. The transportation drift was about 11 meters wide and 7 meters high, and an approximately 8 meters long section of the drift collapsed. Initially, an attempt was made to excavate the material from the collapse in order to enable installation of new support. This was canceled since the removal of broken material only allowed the failure to propagate upwards. Different attempts were made to stabilize the collapse, such as the injection of cement grout into the roof of the failed section and installation of spiling rods to secure the roof. However, none of these attempts made it possible to penetrate through the pile of collapsed material. Eventually, an attempt was made to construct short sections of concrete and steel arches to support the failed material. When a section of arches was built, the material closest to the arch was removed to make room for a new arch section. This process was repeated for about six meters, when continuous the flow of broken rock made it impossible to proceed with the construction of additional arches. The result of the concrete and steel arches is shown in Figure 10.
Since the transportation drift would provide the shortest transport route from orepass 225 to the crushers on the M1250 level, it was decided to try an untested method for penetration of rock piles. During the rehabilitation process, a connecting drift to circumvent the broken material was developed from the side of the transportation drift. Eventually, this drift connected with the transportation drift on the other side of the collapse. On the other side of the pile there was enough room to begin the construction of a heavily reinforced, prefabricated concrete tunnel in front of the pile of collapsed material. The idea was to alternate between pushing the concrete tunnel toward the pile and loading away material in front of the tunnel with loaders moving inside the concrete tunnel (Figure 11).

The concrete tunnel was about 6.8 meters wide and 6.7 meters high, with a total length of about 8 meters. The walls and roof of the tunnel were 0.5 meters thick and heavily reinforced with steel bars. Steel plates covered the outer roof and front edge of the tunnel in order to lower the friction and enhance the durability against wear from the rock material in the pile. Due to the size, the total weight of the whole tunnel to be pushed was about 600 tonnes.

The rock material on the floor of the drift was replaced with sand to decrease the friction between tunnel and floor. The pushing force for the advancement of the tunnel was delivered from four hydraulic jacks placed between the tunnel and two concrete supports that were cast behind the tunnel on both sides and anchored to the rock with rock bolts (Figure 12). Each hydraulic jack delivered a pushing force of about 300 tonnes. Since the jacks had a limited extension range of about one meter, a pre-cast concrete block was placed between the concrete supports and the hydraulic jacks when the tunnel had been moved forward one meter. This approach was repeated for the entire advancement of the concrete tunnel. Low sidewalls were cast on both sides of the tunnel against the rock walls to provide steering and avoid sideway offset during the advancement.
The launching of the concrete tunnel faced some disturbances with, for example, a major inflow of caved rock material at the front. This led to different modifications that eventually resulted in a longer overhang of the tunnel roof at the front, in order to provide a support against falling rocks during the removal of rock material at the front. Before the tunnel reached through the pile of material, an additional support roof was installed in front of the tunnel to obtain a further safety against falling rocks at the front. These measures were successful in the prevention of falling rocks and made it possible to remove enough rock material for the concrete tunnel to be pushed all the way through the collapsed tunnel section.

Concrete walls were casted at the ends of the tunnel between the tunnel walls and the rock walls when the tunnel had reached the desired position. These walls acted as seals when holes were drilled in the roof of the tunnel and concrete was pumped into the cavity above the tunnel to secure the tunnel against falling rock boulders. After the completion of the concrete tunnel advancement the transportation drift was open for the iron ore trucks (Figure 12).
A case study review of the double fatality coal burst at Austar Colliery in NSW, Australia in April 2014

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Abstract

A coal burst occurred on 15 April, 2014 at the Austar Coal Mine, located west of Newcastle, NSW, Australia. The burst resulted in fatal injuries to two men working as part of the mining crew at the development face. At the time, a continuous miner was being used to mine a longwall development gate road through heavily structured coal, at a depth of approximately 550m. A number of pre-cursor bumps had occurred on previous shifts, emanating from the coal ribs of the roadway, in proximity to the coal face.

This paper reviews the geological, geotechnical and mining conditions and circumstances leading up to the coal burst event, and discusses the role and impact of the burst on the roadway stability and the installed rib support in the development heading at the time. The paper also discusses a range of technical and operational considerations of ground support systems needed for operating in such conditions in the future.

1 Introduction

Austar Coal Mine is an underground longwall coal mine located near Cessnock in the Hunter Valley of New South Wales (NSW), Australia and is the only underground mine still extracting the Greta Seam in this region. The mine was the first in Australia to adopt the Chinese-developed Longwall Top Coal Caving (LTCC) method for thick seam extraction. Typically seam thickness ranges from 4m to 7m and depth of mining from 480m to 560m, with future mining planned down to depths of up to 700m, making it one of the deepest operating coal mines in Australia.

On 15 April 2014, a pressure burst occurred in the left hand rib at the active mining face of B Heading, 2 to 3 cut-through, Maingate A9 panel, during development of the gates roads for the ninth longwall top coal caving panel. Strata in the general vicinity was affected by disturbed geology and multiple geological structures. Figure 1 shows a section of the mine plan as at October 2014 (six months after the accident), indicating the current longwall extraction panel (A8) and the development panel A9 where the accident occurred. (Note: Neither development nor longwall face positions changed significantly between the time of the accident and the date of this plan). At the time of the accident the current longwall face was in excess of 1,000m away from the Maingate A9 development panel face position.

When the accident occurred, the development face was being advanced by a crew of seven mine workers. Messrs Jamie Mitchell and Phillip Grant were located on a working platform on the left hand side of the ABM25 continuous miner (bolter-miner), immediately adjacent to a ribline that had already been supported with bolts and mesh. The two men were engulfed by material ejected from the ribline during the pressure burst and died at the scene.

The accident was reported by the NSW Mine Safety Investigation Unit (MSIU 2015) following an extensive investigation, which included a detailed technical report prepared by Galvin & Hebblewhite (2015).
1.1 Terminology

In any situation, four conditions have to be satisfied simultaneously in order for a dynamic (violent) rock failure to occur. The first is self-evident and implicit in the other three conditions reported by Salamon & Wagner (1979). These four conditions are:

1. The stress environment must be sufficiently high to result in rock failure.

2. A situation must exist which can result in a state of unstable equilibrium. This could be a low friction bedding plane, for example, where the potential exists for the coefficient of friction to drop rapidly from its static to dynamic value once movement is initiated along this plane.

3. A change in the loading system. Potential triggers include, for example, a reduction in system strength due to a local change in rock mass material or structural properties; an increase in system stress associated with a local geological structure; or a decrease in confinement due to the formation of one or more excavations.

4. A large amount of energy has to be stored in the system. This energy can be generated, for example, by depth of mining, bridging strata or geological structures.

Whilst these conditions were applied by Salamon and Wagner to rock burst behaviour in hard rock mines, they are also potentially applicable to similar dynamic, stress-driven events in underground coal mining.
There are a number of terms used across the international underground mining industry (including both hard and soft rock mining) that are of relevance to any discussion of dynamic ground failure events in underground mines, including the type of event that occurred at Austar Coal Mine on 15 April 2014. It is important to clarify and adopt a consistent set of terms used in the context of such dynamic rock failures. It must be recognised that there is no universally accepted and unique set of definitions for all of these terms, however the following descriptions are widely regarded as appropriate – at least within the Australian mining context.

The terms to be discussed are as follows:

- Rock burst
- Strain burst
- Pressure bump
- Pressure burst
- Shake-down
- Outburst
- Coal bump
- Coal burst
- Pillar bump
- Pillar burst

All of the above describe events associated with some form of dynamic energy release, usually associated with intact rock failure. This release of energy can vary greatly in magnitude and may or may not generate a measurable seismic signal.

**Rock bursts** and **strain bursts** are terms used to describe such dynamic energy releases and rock failure associated with hard rock mining. The source of the energy is directly related to stress levels within the rock, albeit that the manifestation of the stresses, and the triggers for the release of the energy can be quite complex, involving many factors. The difference between a rock burst and a strain burst is simply one of consequence scale, due to different energy magnitudes – with strain bursts being of much lower energy magnitude, such that the resulting rock damage is far less than for a typical rock burst. These terms are not generally used in underground coal mining, although the geotechnical mechanisms involved may be very similar to the coal mining equivalent events summarised below.

The next two terms are those most commonly used to describe dynamic energy releases in underground coal mining – **pressure bumps** and **pressure bursts**. Both terms refer again to dynamic energy events associated with stress levels in the rock mass, which includes but is not limited to the coal seams. However, the commonly accepted difference between a pressure bump and a pressure burst relates to the magnitude and, hence, consequence. A pressure bump is a dynamic release of energy within the rock (or coal) mass in a coal mine, often due to intact rock failure or failure/displacement along a geological structure, that generates - an audible signal; ground vibration; and potential for displacement of existing loose or fractured material into mine openings. (A pressure bump is also sometimes referred to as a bounce). On the other hand, a pressure burst is a pressure bump that actually causes consequent dynamic rock/coal failure in the vicinity of a mine opening, resulting in high velocity expulsion of this broken/failed material into the mine opening. The energy levels and, hence, velocities involved here can cause significant damage to, or destruction of conventional installed ground support elements such as bolts and mesh. A **shake-down** is another term taken from the hard rock mining sector, referring to damage caused by a bump event, where existing broken rock material is destabilised and collapses into the mine excavation.
An outburst in Australian mining terminology is also a dynamic energy release that can lead to some form of rock failure, however the source of energy is primarily associated with in situ gas pressure, sometimes also supplemented by stress-related energy. Therefore, outbursts are normally only associated with coal mining (where there is more prevalence of in situ gas), and usually only occur within the coal seam. Caution is emphasised with the use of the term ‘outburst’, when reviewing international literature. Whilst most European deep coal mining industries adopt the terminology as described above, the US coal industry often uses the term ‘outburst’ more broadly, to describe dynamic events that are purely stress driven as the energy source – events that in Australia would be referred to as a pressure burst or a coal burst.

The terms coal bump and coal burst; together with pillar bump and pillar burst are generally synonymous with pressure bump and pressure burst - and are all terms used to describe such dynamic events in underground coal mining. These terms are in some ways an alternative name for a sub-set of the more general events covered by pressure bump and pressure burst. Coal bumps and bursts are specific to events emanating from within the coal seam (as opposed to roof or floor origin); while pillar bumps and bursts relate to events within pillars as opposed to either in solid development drivage or on a longwall face, for example.

1.2 Mining Background

The following background information is provided regarding the mining operations at Austar Coal Mine (Austar). Austar is a deep underground coal mine located approximately 10 km southwest of Cessnock in the Newcastle Coalfields of New South Wales, Australia. It is owned by Yancoal Australia Limited, an Australian-Chinese partnership. Yancoal purchased the mine (formerly known as Southland Colliery) in December 2004. Austar commenced mining operations in April 2005 and in September 2006 became the first mine in Australia to adopt the mining method called Longwall Top Coal Caving (LTCC), with the technology based on Chinese experience. The LTCC operations at Austar, and associated geotechnical challenges, were described by Moodie and Anderson (2011). Figure 2 is a simplified schematic of the LTCC face and surrounding caving behaviour. Longwall development for LTCC is conducted in the same way as for conventional retreat longwall faces, using a two-heading gate road development configuration, mined using a single-pass bolter-miner and shuttle cars.

![Figure 2 Schematic view of LTCC face and caving behaviour (after Moodie & Anderson 2011)](image)

Austar mines the Greta Seam at depths approaching 600m, with plans to proceed to greater depths (towards 700m) within the medium-term future. At the time of the incident, the depth being mined was 555m. The Greta Seam has had a chequered history associated with difficult mining conditions due to high stresses, gas and other high risk factors. There is a long history of pressure bumps associated with mining in...
the Greta Seam at Austar and other operations. Much of this is anecdotal, particularly within mine management circles. Large and frequent bumps have been experienced in previous neighbouring mines, in first workings at a depth of around 300 m, especially when mining through geological structures. Previous mine managers from the region consistently reported that rib spall was associated with bumps. There was some uncertainty as to whether some bumps may have constituted coal bursts under the definitions above. It was considered by the majority of experienced senior mine personnel in the region that bumping was ‘normal’ in the Greta Seam and could be quite severe.

Austar has experienced regular bump activity during the past five to ten years. In fact, the general consensus amongst the mining workforce was that bumping was “normal” and to be expected, and served to relieve high stress concentrations in and around the face – in this sense, regular bumping was regarded as a good thing, a “pressure relief” mechanism. In 2011 Austar experienced a major bump and shake-down event in the 300 Mains development panel. This incident involved the loss of the sidewall of the roadway for about 50 to 60 m, resulting in the sidewall needing to be resupported.

In spite of this experience, the consensus of the technical staff at Austar at the time, and prior to the accident, was that the coal seam characteristics were not prone to pressure bursts, as the structured nature of the coal would prevent sufficient stress to be stored in the coal to cause a burst. This view is in contrast to extensive international experience suggesting that virtually all coals can burst under certain circumstances, and that the physical and mechanical properties of coal are not necessarily key factors in determining propensity to bursting (Bräuner 1994, Iannacchione & Zelanko 1995, Mark 2014).

2 Geotechnical environment

It is well known that the immediate roof at Austar (and some floor strata) can include a number of massive sandstone strata units which are then overlain by the massive sandstone Branxton Formation in the upper roof. In terms of structural geological features, Figure 1 indicated the presence of a major zone of faulting known as the Quorrobolong Fault Zone, which ran to the east, and roughly parallel to the 300 Mains development panel. This zone of faults intersected each of the longwall development panels, including Maingate A9 Panel, which is shown in close-up detail in Figure 3.

The significant extent and throw of faulting between cut-throughs 1 and 2 resulted in very slow development progress through this region (over six months), and included an amount of stone drivage using a roadheader. Prior to the accident, an additional, unexpected inbye fault was intersected in A Heading resulting in a roof fall. As a result of this fault being encountered in A Heading, a decision was taken to mine an angled stub-heading off 2 cut-through below B Heading in an attempt to prove the fault location prior to advancing B Heading. As this angled stub was extended, a significant sheared zone was encountered and another roof fall occurred, although it is not clear if there was also a fault present just beyond this fall.

The mine was well aware of the presence of the initial zone of regional faulting between 1 and 2 cut-throughs. In addition to the larger scale structural geological features, visual inspection of the rib conditions (evident in photographs taken of the site (NSW MSIU 2015)) revealed some quite variable rib characteristics, which suggested localised potential variation in cleat distribution impacting on rib behaviour.
3 Accident circumstances

3.1 Pre-cursor experience

Pressure bumps were quite commonplace during the development of the previous gateroads in Maingate A8 Panel – both during the intersection of the Quorrobolong Fault Zone, and also beyond it. This experience was repeated during the initial Maingate A9 development. The following is an abbreviated description of events prior to the accident, summarised from the Accident Investigation Unit Report (NSW MSIU 2015).

During the course of the shifts immediately preceding the accident, a number of pressure bumps were reported by the deputies in the panel, with these bumps occurring especially during the process of cutting coal.

The shift 24 hours before the incident was an afternoon shift with the same crew that was present when the accident occurred. On this shift the deputy reported a very large pressure bump emanating from the right hand rib in B Heading while cutting was taking place. The bump was of such intensity that a decision was taken by the deputy and the miner driver to stop bolting while cutting, so that all bolting (roof and rib) would be completed prior to commencing the next cutting sequence. The shift directly preceding the incident was the day shift of 15 April 2014. There was no production on this shift as it was a planned maintenance shift. There is no record of any pressure bumping during day shift.

The afternoon shift of 15 April 2014 began at 3 pm. The incident occurred at 9.05 pm in B Heading inbye of 2 cut through. Mr Mitchell and Mr Grant were both on the left-hand side of CM 35. The operator of CM 35 had just completed loading...
the shuttle car and had raised the cutting-head to the roof to park the cutting head. As soon as he turned off the conveyor, the incident occurred. Approximately 38 cubic metres of coal was ejected from the left-hand side wall. The exact force and speed of the ejection is not known, but it is clear from the eye witness accounts that the coal was ejected from the side wall with significant force. The deputy in charge of MG A9 B crew (the deputy) was driving the shuttle car at the time of the incident and describes the event as follows:

“It was like there was an explosion. It was, there was massive, I was sitting in the shuttle car, it blew me into the mesh guarding on the shuttle car. I lost my helmet. When I sort of, after it, it was like a split second, it was just that quick and that intense and it just, I sort of gathered myself.”

3.2 Coal burst event

Mr Mitchell and Mr Grant were both on the left hand side of the work platform on board the continuous miner, directly adjacent to where the rib burst occurred and were buried as a result of the burst. Figure 4 is a plan of the accident site, indicating the burst location on the left hand rib, extending from a position just over a metre back from the face, to over 12m back from the face. The rib in this location had been bolted and meshed. The depth of the burst into the rib is not known accurately, but it certainly extended well beyond the depth of the installed rib bolts (1.5m and 2.1m bolts) which were fully ejected amongst the broken coal.

![Plan of accident site in B Heading, MG A9 Panel](image)

Figure 4 Plan of accident site in B Heading, MG A9 Panel (after NSW MSIU 2015)

Figure 5 shows the roadway conditions in the heading at the time of the burst. Figure 5 is a view taken from the rear of the shuttle car that is parked directly behind the continuous miner at the face. The burst coal
can be seen on the left of the miner, where the installed mesh is displaced outward, beneath the ventilation ducting. Figure 6(a) and Figure 6(b) are close-up photographs of the actual burst site in the left hand rib, with 6(a) showing the burst coal adjacent to the continuous miner (after body recovery) and (b) being closer to the face towards the right hand end of the burst. Figure 7 shows the damaged continuous miner, with clear evidence of the impact of the burst coal on the miner platform hand rails, in the vicinity of where the men had been standing.

Figure 5  B Heading showing the continuous miner and shuttle car at the burst site (left hand side of CM) *(after NSW MSIU (2015))*
Figure 6(a) Burst site adjacent to continuous miner (after recovery of bodies) (after NSW MSIU 2015)

Figure 6(b) Inbye end of burst cavity showing the distinctive Dosco Band which formed an upper bound to the burst (after NSW MSIU 2015)
A number of significant features are evident in these images. Firstly, the collars of some of the previously installed rib bolts can be seen displaced amongst the loose coal material (in Fig. 6(a)); secondly, the sizing of the coal that has been expelled from the burst is quite variable, ranging from well fragmented small particles, to a number of larger blocks; thirdly, the upper bound of the burst cavity is clearly visible (Fig. 6b) as a very smooth, flat bedding plane within the seam known as the “Dosco Band”. Rib coal above the Dosco band has not failed or displaced at all, whereas all the coal beneath it is part of the burst. The exposed surface of the Dosco Band showed signs of horizontal shearing activity, with a quite distinctive reddish-brown dust coating on much of the surface. Newman (2002) and others have previously reported similar evidence of reddish-brown pulverised coal particles at burst sites.

4 Rib support performance

As indicated earlier, the installed rib support at the accident site consisted of 1.5m mechanically anchored bolts in the lower and upper sections of the seam, supplemented by 2.1m chemically anchored bolts in the mid-seam section. This represented the standard Code Yellow rib support plan for the mine (see Figure 8). All bolts had been installed at or near the face position, prior to the event, and the ribs were also fully meshed.

Figures 9(a) and 9(b) show typical bolted and meshed rib conditions in the MG A9 development panel, prior to the accident. These illustrate the highly loaded rib conditions with evidence of some buckling failure; the heavily cleated nature of the ribs; and some apparent mining-induced fracturing in the rib surfaces (see Fig. 9(b)). The dominant horizontal “Dosco Band” bedding plane below the roof horizon is also evident in Figure 9(a). Also evident in Figure 9(b) is some additional installed rib support, over and above that prescribed by Code Yellow. The mining crews had commenced installing extra rib support prior to the accident, due to the level of bumping and rib deterioration that was occurring.
Figure 10 shows some of the previously installed rib bolts extracted from the burst muckpile. These indicate that the bolts were expelled fully by the burst, with no anchorage capacity extending beyond the burst cavity; and also show evidence of almost full encapsulation of the bolts, indicated by the resin column on each bolt.

In terms of the actual burst event, and the role of the installed rib support, the following conclusions can be drawn:

5. The rib bolts at the site (and mesh) appear to have been installed according to the required support rules prior to the burst, and the extent of resin encapsulation was as planned.
6. The depth of the burst cavity was well beyond the rib bolt length.
7. The size distribution of the coal particles expelled by the burst was predominantly quite small, indicating that extensive failure occurred not only beyond the bolt length, but also between the bolts.
8. There was no evidence of any failure of coal above the Dosco Band, which clearly acted as a failure plane boundary on which significant dynamic shear failure occurred.

If the question is then posed – “could additional or different support strategies have prevented this burst?”, it is difficult to answer definitively, due to the complexity and dynamic nature of the failure mechanism. Longer bolts or more closely spaced bolts may have changed the rib behaviour, but would probably not have been sufficient to prevent the burst occurrence. Angled bolts that penetrated the Dosco Band may also have been able to influence the extent of shear displacement occurring on that horizon, but the forces involved in the failure were probably too great to be contained by a small number of bolts or cables, so this option is also not considered a feasible control measure in isolation.
A case study review of the double fatality coal burst at Austar Colliery in NSW, Australia in April 2014

B. K. Hebblewhite and J. M. Galvin

Figure 8  Rib Support Rules in use at accident site (after NSW MSIU 2015)
Figure 9(a) Outbye rib conditions in MG A9 Panel (A Heading) *(after NSW MSIU 2015)*

Figure 9(b) Rib conditions outbye from the burst site *(after NSW MSIU 2015)*
Possible failure mechanisms – contributing factors

There is no doubt that the mechanics of what causes a pressure burst, and what are the contributing factors, is extremely complex. There remains a considerable amount of research effort to be applied in the future to this complex and dynamic rock failure behaviour – in all its different manifestations. However, there is extensive international experience already available. International experience with bumps and bursts suggests that most such events are the result of multiple different mechanisms and factors coming together.

In the case of the Austar event, the following factors were present – all of which could potentially have contributed to the accident:

- High stress, associated with the depth of mining, and possibly supplemented by some additional stress concentrations resulting from any or all of:
  - Regional faulting and shear zones immediately adjacent to the event location;
  - Lensing and variations in stiff overburden sandstone units (and possibly also floor units).
- The presence of quite intense regional geological structure in the area, combined with severely distorted and complex local geology and cleat patterns.
- The presence of massive sandstone units within the immediate 20m+ of overburden roof, and the possibility of massive units also in the floor.
- A very dominant, smooth horizontal shear plane represented by the Dosco Band, providing a dynamic shear failure surface below which the crushed and sheared coal could move.
- The effect of development mining providing a trigger either to destabilising the rock material above the burst zone in proximity to the fault surfaces ahead of mining, and/or providing a loss of confinement to the highly stressed coal in the rib that was undoubtedly subject to high levels of vertical stress.
6 Conclusions

It is not possible on the evidence available to categorically state the precise cause(s) of the pressure burst that occurred at Austar on 15 April 2014, nor to state the relative magnitude or significance of the contributory factors, with a high degree of certainty. However, it is clear that a range of geological and geotechnical parameters all contributed to this event, together with a change in mining-induced stresses associated with removal of confinement simply by the incremental process of development mining. The installed level of rib support appears to have been in accordance with design requirements, but has not been able to contain the burst, with failure extending well beyond the depth of the installed rib bolts.

In terms of future prevention and/or control measures, the ultimate measure is to move towards fully automated mining to ensure personnel are not located in the vicinity of the development roadway faces. However, this remains an elusive goal, at least for the short to medium term.

Therefore, what is needed is a combination of:

a. ongoing investigative research into potential burst mechanisms and prediction and control measures; and

b. a comprehensive strategy of measures by mine operators to identify zones of potential burst-prone conditions, such as those characterised by disturbed structural geology (both large and small scale features); zones where stress concentrations may occur due to either geological variations or mining activity; zones where bumping and related seismic activity is present, to potentially provide a greater understanding of bump/burst behaviour and also to serve as a locator of high risk zones. On the basis of such burst-potential zone identification, there is then a need to develop modified remote-mining practices and ground support regimes for such zones, in order to minimise the risk of injury or disruption.

References


A follow up to the behaviour of cable bolts in shear; Experimental study and mathematical modelling

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Abstract

A mathematical model is developed to determine the pure shear strength of cable bolt under various shearing conditions. The proposed model is developed based on the Fourier series concept and a linear relationship between shear and normal forces generated during the cable bolt shearing. The conventional double shearing apparatus (MKII) was modified to evaluate the pure shear strength of cable bolt by removing the contribution of concrete blocks contact frictional force. The experimental results obtained from the modified double shear system (MKIII) were in good agreement with the proposed model prediction.

1 Introduction

British Standard (BS 7861-2: 1996) since its development has been widely incorporated to determine shear strength properties of various tendons used in mining and civil industries. This standard indeed replicates guillotine box shearing whereby the bolt is cut across its cross section without being encapsulated in the host strata. As shown in Figure 1, the bolt in the passive mode (i.e. no pretension) was positioned inside the apparatus and loaded vertically until the pronounced failure was observed.

Despite of the fact that the British Standard can be carried out easily and without cumbersome initial sample preparation work, it possesses some conspicuous drawbacks as:

- The shear strength is underestimated as the bolt is not pretensioned prior to shearing,
- The testing assembly inevitably will lead to a metal to metal shearing (Figure 2),
- The bolt is not encapsulated inside the host strata, resulting in different confinement stiffness as compared to the field conditions.

Figure 1 Sectional diagram of double embedment shear frame with the unit being tested (BS 7861-2: 1996)
Aziz et al., (2003) incorporated the concept of double shearing to investigate the shear strength of three common types of bolt used in Australian mining industries. The instrument (MKI) that was developed in this testing program is shown in Figure 3. Unlike the single shear test of British Standards (BS 7861-2: 1996), the bolt was encapsulated inside the concrete blocks, representing the host strata and pretensioned prior to shearing.

Aziz et al., (2005) continued the previous study to investigate the effect of resin thickness in shear for bolt-grout-concrete interaction by double shear testing. ANSYS commercial software was applied to simulate numerically the experimental results obtained from this study.

Craig and Aziz (2010) studied shear behavior of cable bolts using a large scale double shear instrument (MKII) as shown in Figure 4. It consisted of three concrete blocks with outer two cube blocks of 300 x300 and 300 mm sides and the middle block of 300 x 300 x 450 mm sides. The concrete blocks were cast in the steel frame of the double shear apparatus, and before the concrete could be cast, a 20 mm diameter
conduit pipe was placed through the pre-cut holes in the centre of the wooden ends and galvanised steel separators of the mould. The cable bolt was eventually encapsulated in the concrete blocks using an appropriate grout.

Figure 4  Large scale double shear instrument

An experimental study on the shear performance of plain and spiral cable bolts was carried out by Aziz et al., (2014) using double shear apparatus. Other studies on shear behavior of cable bolts incorporating the same experimental instrument, were reported by Aziz et al., (2015a), Rasekh et al., (2015) and Li et al., (2015a) and (2015b).

An original mathematical model for the shear behavior of cable bolt was introduced by Aziz et al., (2015b). The model was associated with a set of systematic experimental study. Experimental study was performed and results were compared with the proposed mathematical equation.

The above mentioned mathematical equation designates the combination of shear strength of cable bolt and frictional force generated due to concrete block sliding. Since then the model has been further developed which calculates only the pure shear strength of cable bolt. This aspect of the study together with modified double shear equipment is the subject of this paper.

2  Mathematical modeling

The mathematical model is based on the assumption of a linear relationship between the shear and normal loads as:

\[ S - N \tan(\varphi) - c = 0 \]  \hspace{1cm} (1)

where:

\[ \begin{align*}
    S & = \text{shear load}, \\
    N & = \text{normal load}, \\
    \varphi & = \text{friction angle}, \\
    c & = \text{cohesion}.
\end{align*} \]
The Fourier series concept is applied to replicate the variation of the normal load against shear displacement. Fourier series is a mathematical technique incorporated to solve a large variety of engineering problems mainly adopting the principle of superposition:

\[ N = \frac{a_0}{2} + \sum_{n=1}^{N} a_n \cos\left(\frac{2\pi n u}{T}\right) + b_n \sin\left(\frac{2\pi n u}{T}\right) \]

\[ a_n = \frac{2}{T} \int_{0}^{T} \sigma_n \cos\left(\frac{2\pi n u}{T}\right) du \]

\[ b_n = \frac{2}{T} \int_{0}^{T} \sigma_n \sin\left(\frac{2\pi n u}{T}\right) du \]

where:
- \( a_n \) and \( b_n \) = Fourier coefficients,
- \( N \) = number of Fourier coefficient,
- \( u \) = shear displacement,
- \( T \) = shearing length.

Introducing Equations (2a, b and c) in equation (1) by considering \( a_0 \) to \( a_3 \), the shear strength is obtained as:

\[ S = \left( \frac{a_0}{2} + \sum_{n=1}^{3} a_n \cos\left(\frac{2\pi n u}{T}\right) \right) \tan(\varphi) + c \]

The shear displacement at peak shear strength is determined by taking derivation of the above relationship with respect to the shear displacement and equating to zero as:

\[ \frac{d\left\{ \frac{a_0}{2} + \sum_{n=1}^{3} a_n \cos\left(\frac{2\pi n u}{T}\right) \right\} \tan(\varphi) + c}{du} = 0 \]

Thus, the peak shear displacement at peak shear strength \( (u_p) \) is obtained as:

\[ u_p = \frac{T}{2\pi \cos^{-1}\left\{ \frac{-4a_2 + \sqrt{16a_2 - 48a_1a_3 + 144a_3^2}}{24a_3} \right\}} \]

Introducing equation (5) in equation (3), the peak shear strength \( (S_p) \) is proposed as:

\[ S_p = \left( \frac{a_0}{2} + \sum_{n=1}^{3} a_n \cos\left(\frac{2\pi n u}{T}\right) \right) \tan(\varphi) + c \]

The model coefficients including Fourier coefficients \( (a_n) \), cohesion \( (C) \) and angle of friction \( (\varphi) \) are determined according to the measured data for various test conditions such as the cable type and pretension.

Equation 6 determines the total shear strength of reinforced concrete blocks. This consists of cable bolt shear strength and the additional shear force generated by the concrete surface friction. In order to obtain
the pure shear strength of the cable bolt, the frictional term should be quantified and subsequently removed from the total shear strength.

The frictional force generated in the process of shearing follows the Coulomb tribological equation as:

\[ S = N \tan(\varphi_b) \]  

(7)

where:

\( \varphi_b \) - concrete surface basic friction angle determined by tilt testing.

Deducting equation 7 from equation 6, the pure shear strength of cable bolt \( S_p^b \) is obtained as:

\[ S_p^b = \left( \frac{d_0}{2} + \sum_{n=1}^{3} a_n \cos\left(\frac{2n \pi T}{2\pi} \right) \right) \left[ \frac{-4a_2 + \sqrt{16a_2 - 48a_1a_3 + 144a_1^2}}{24a_3} \right] \left[ \tan(\varphi) - \tan(\varphi_b) \right] + c \]

(8)

3 Concrete surface basic friction angle

Double shearing test (Aziz et al., 2015b), without cable bolt as the reinforcing element, was carried out to determine the concrete surface basic friction angle. The normal load subjected to concrete blocks started with 50 kN and increased incrementally every 20 mm, reaching to 250 kN at the end of the test. The value of shear load against shear displacement was measured and subsequently incorporated to calculate the concrete surface basic friction angle as shown in Figure 5. The basic friction angle was indicated as 26.94°.

By introducing the value of basic friction angle in Equation 8, the pure shear strength of cable bolt is obtained as:

\[ S_p^b = \left( \frac{d_0}{2} + \sum_{n=1}^{3} a_n \cos\left(\frac{2n \pi T}{2\pi} \right) \right) \left[ \frac{-4a_2 + \sqrt{16a_2 - 48a_1a_3 + 144a_1^2}}{24a_3} \right] \left[ \tan(\varphi) - \tan(26.94^\circ) \right] + c \]

(9)
4 Model calibration and verification

The coefficients in Equation 9 were calibrated for various conditions of cable type, pre-tension value and bonding agent incorporating the experimental data reported by Aziz et al., (2015b) and listed in Table 1.

Two additional double-shear tests were undertaken to verify the proposed equation for the pure shear strength of cable bolts. In one test the concrete surfaces were maintained in contact with each other and in the other without (that is no frictional resistance). In both tests various parameters, such as pretension load, grout type and concrete strength were kept constant. To achieve cable bolt shearing without contact between concrete blocks, the double shear apparatus was modified by installing two lateral braces on each side of the assembly to prevent normal load on the concrete blocks during shearing as shown in Figure 6a. A pair of Teflon sheets with negligible friction coefficient was introduced between concrete joints surfaces of the modified apparatus MKIII to further assure no friction between concrete blocks, as shown in Figure 6b. Table 2 compares the double-shear values of the pure shear strength of cable bolts obtained from the proposed equation and experiments. It is clear that the experimental test result is reasonably close to the prediction from the mathematical equation.
## Table 1a  Tested cables and the test environment Aziz et al., (2015b)

| Test No. | Product name | Cable diameter (mm) | Wire geometry | Wire geometry | Cable cross-section | Wire geometry | Cable cross-section | Bonding agent | Drill bit (mm) | Bonding agent | Bonding agent | Bonding agent | Bonding agent |
|----------|--------------|---------------------|---------------|---------------|---------------------|---------------|---------------------|---------------|---------------|---------------|---------------|---------------|---------------|---------------|
| 1        | Superstrand  | 21.8                | Spiral        | 19 wire, PC strand | Non-birdcaged       | 28             | Oil based resin     | 250           | 558           |               |               |               |               |
| 2        | Superstrand  | 21.8                | Plain         | 19 wire, PC strand | Non-birdcaged       | 28             | Oil based resin     | 250           | 628           |               |               |               |               |
| 3        | TG           | 28                  | Spiral        | 9 wires, hollow centre | Non-birdcaged       | 42             | TD80 Grout          | 250           | 604           |               |               |               |               |
| 4        | SUMO         | 28                  | Spiral        | 9 wires, hollow centre | 35mm birdcage       | 42             | TD80 Grout          | 250           | 414           |               |               |               |               |
| 5        | SUMO         | 28                  | Spiral        | 9 wires, hollow centre | 35mm birdcage       | 42             | TD80 Grout          | 100           | 488           |               |               |               |               |
| 6        | Plain SUMO   | 28                  | Plain         | 9 wires, hollow centre | 35mm birdcage       | 42             | TD80 Grout          | 250           | 711           |               |               |               |               |
| 7        | Plain SUMO   | 28                  | Plain         | 9 wires, hollow centre | 35mm birdcage       | 42             | TD80 Grout          | 100           | 659           |               |               |               |               |
| 8        | Gardford twin-strand | 15.2 | Plain | 2 x 7 wire, PC strand | 25mm Bulbs | 55 | BU100 Grout | 0 | 501 |               |               |               |               |               |

## Table 1b  Model coefficients for different types

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<td>-75.64</td>
<td>55.49</td>
<td>59.56</td>
<td>12.7</td>
</tr>
<tr>
<td>7</td>
<td>449.78</td>
<td>-136.34</td>
<td>16.39</td>
<td>-3.91</td>
<td>61.33</td>
<td>0.44</td>
</tr>
<tr>
<td>8</td>
<td>235.38</td>
<td>-157.50</td>
<td>42.83</td>
<td>-4.21</td>
<td>47.61</td>
<td>138</td>
</tr>
</tbody>
</table>
Table 2a  Comparison between the shear strength values of cable bolts with and without friction between concrete blocks

<table>
<thead>
<tr>
<th>Test No.</th>
<th>Product type</th>
<th>Bonding agent</th>
<th>Pre-tension load (kN)</th>
<th>Peak shear load per face (kN)</th>
<th>Friction between surfaces</th>
</tr>
</thead>
<tbody>
<tr>
<td>9</td>
<td>Plain</td>
<td>Strata binder HS</td>
<td>5</td>
<td>645.64</td>
<td>with</td>
</tr>
<tr>
<td>10</td>
<td>Plain</td>
<td>Strata binder HS</td>
<td>5</td>
<td>442.16</td>
<td>without</td>
</tr>
</tbody>
</table>

Table 2b  Determination of shear load without friction between concrete blocks by the model for plain superstrand cable with 5 kN of pretension load*

<table>
<thead>
<tr>
<th>Test type</th>
<th>a0</th>
<th>a1</th>
<th>a2</th>
<th>a3</th>
<th>Model normal load (kN)</th>
<th>tan φ</th>
<th>tan(26.94°)</th>
<th>c (kN)</th>
<th>Measured peak shear load per face (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Plain superstrand</td>
<td>324.77</td>
<td>182.37</td>
<td>18.65</td>
<td>3.04</td>
<td>366.3</td>
<td>1.47</td>
<td>0.508</td>
<td>88.6</td>
<td>441</td>
</tr>
</tbody>
</table>

*Fourier coefficients were determined using Equation 2 and friction angle and cohesion were calculated by plotting the variation of shear load against normal load after the elastic stage.

5  Conclusion

- A new mathematical model is developed to calculate pure shear strength of cable bolts. The model was tested against the experimental data. The calculated shear failure load was in close agreement with the experimental test results. The model can also be used for other types of tendons used for ground reinforcement.
- The modified double-shear apparatus (MKIII) is capable of determining pure shear strength of the tendons alone. An initial test was undertaken using a cable bolt and further experimental studies are planned for testing of different marketed cables in Australia for both civil and mining engineering applications.

References


A numerical modelling case study - correlation of ground support instrumentation data with a three dimensional inelastic model

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P.M. Dight, University of Western Australia, Australia
Y. Potvin, University of Western Australia, Australia

Abstract

As a sub-project of the industry-sponsored initiative GSSO (Ground Support Systems Optimisation Project) being delivered by the Australian Centre for Geomechanics (University of Western Australia), a numerical modelling exercise has been carried out using ground support instrumentation and rockmass data acquired at George Fisher Mine (part of Glencore’s Mount Isa Mines operation) in Mount Isa, Queensland, Australia. The aim of the modelling was to attempt to correlate field measurements with model results, using a commercially available numerical modelling package with realistic rockmass parameters. The over-arching aim of the project is to assess the limits of applicability of numerical modelling in ground support system design. The three dimensional inelastic code FLAC3d (Itasca) was selected due to its capability to model a realistic mine-wide geometry while providing sufficient detail in the instrumentation sites’ areas of interest. FLAC3d also has the constitutive models and support element formulations which make it possible to incorporate the field measurements acquired during the monitoring campaign. Results in terms of excavation closure, cable loads and fibrecrete load correlation are presented and discussed. Implications for ground support system design are raised.

1 Introduction

The Australian Centre for Geomechanics (ACG) commenced the industry funded research project “Ground Support Systems Optimisation” (GSSO) in late 2013. The project objective is to explore whether it is possible to optimise ground support systems, with the aim to maintain, if not improve mine safety and economics. The project comprises three sub-projects, namely: 1) probabilistic ground support design; 2) the use of numerical modelling for ground support design; and 3) benchmarking of current ground support practices. This paper focusses on sub-project 2 and features a numerical simulation of an underground instrumentation project with the view to correlating the results. The over-arching goal is to determine whether it is practical to design ground support systems using commercially available numerical codes.

The instrumentation trial took place at GSSO sponsor site George Fisher Mine and is described in another paper in these proceedings, entitled: “An instrumentation project to investigate the response of a ground support system to stoping induced deformation” (Sweby, et. al. 2016).

2 Objectives

The objectives of the numerical modelling component of the GSSO were as follows;

1. To simulate, as close as practical, the mining geometry of the trial sites: the trial sites are both located in a footwall drive on the 17 level, as shown in Figure 1. From this it can be seen that the geometry at both sites is complex and cannot be approximated by a two-dimensional (plane strain) approach. Thus to satisfy this objective, a three dimensional numerical code was required.
2. To incorporate the influence of development and stoping voids outside of the trial sites as accurately as practical. In order to capture the stress redistribution due to stoping, a modelling application capable of incorporating the regional stoping effects, without compromising computing efficiency, was required.

3. To use a material behaviour model appropriate to the prevailing rockmass conditions: The characteristics of the rockmass in which the instrumentation sites are located are best demonstrated in Figure 2. The rockmass is a closely interbedded sequence of brittle, moderately strong phyllites and sandstones.

Figure 1  Location of the instrumentation trial sites on the 17 level

Figure 2  Closely spaced bedding and cross jointing evident in this pillar exposure (rockbolt head highlighted for scale). (Photograph provided by George Fisher Mine)
4. To incorporate sequential excavation and support for the trial sites: to accurately simulate the loading path and deformation history of the rockmass at each of the sites, it was a requirement of the model that the sites be excavated (and supported) in a sequence that closely matched reality.

5. To accurately model the installed SMART cable instrumentation: software requirements necessitate the inclusion of a support element capable of accurately simulating the deformation and load characteristics of the installed SMART instruments.

6. To incorporate liner elements for correlation with strain measurements: as above, a liner element capable of capturing the key component measured, i.e. circumferential strain.

7. To accurately track excavation deformation for correlation with extensometer measurements.

3 Numerical model

To address all of the modelling objectives highlighted above, the inelastic finite difference code FLAC3d (Itasca, 2012) was chosen as the most appropriate commercially available numerical modelling tool.

Stope and development shapes were used to generate detailed meshes from photogrammetry survey data in the region of the two instrumentation sites, using the 3d modelling software Rhinoceros® (McNeel & Associates, 2016) and the Itasca software KUBRIX-Geo (Itasca, 2015). The mesh detail is shown in Figure 3, for the north site.

Excavations outside the trial sites were modelled using the Octree meshing logic in Flac3d, which densifies the rockmass surrounding excavations into a graded mesh approximating the excavation shape. An example is shown in Figure 4.

The entire model, including all the excavations to be modelled, is shown in Figure 5, with approximate dimensions for scale.

![Flac3D Mesh detail for the north instrumentation site](image-url)

Figure 3 Flac3D Mesh detail for the north instrumentation site (shown circled in red on left image)
Figure 4  Octree meshing detail for stopes and development remote from the trial sites. On the left is shown a 3D solid representation of a stope (grey) with the mesh subdivision in red.

Figure 5  Flac3d model showing all excavations to be modelled (blue – development, green – stopes, cyan – upper stopes). Approximate model dimensions (length and depth) are shown for scale.
4 Model inputs

Based on the laminated nature of the rockmass (Figure 2), the constitutive model initially selected for simulating material behaviour was the Ubiquitous Joint (UJ) model. This model accounts for the presence of weakness planes within a Mohr-Coulomb material model (Itasca, 2012). The criterion for slip on the plane of given orientation consists of a composite Mohr-Coulomb envelope with tension cut-off. In this model, general failure of the ‘matrix’ is detected and plastic corrections applied. The resulting stresses are then analysed for slip on the weakness plane, a plastic flow rule applied and updated accordingly.

The UJ model was applied to the region surrounding the instrumentation sites, to a distance of approximately 20m away from the excavation centreline. Joint and rock properties assigned to the UJ model are listed in Table 1 (based on laboratory testing but varied in order to match the instrumentation data). Elastic model properties were assigned to the remainder of the model, with properties as given in Table 1.

The orientation of the bedding at the instrumentation sites was determined by field mapping (compass) and photogrammetry (Adam Technology, 2016) and values for dip and azimuth of 55° and 285° respectively were applied (Figure 6).

Correlations between closure measurements derived using the UJ model are currently a work-in-progress and further development of the post-failure strength parameters is required.

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Young’s Modulus</td>
<td>9.3 GPa</td>
</tr>
<tr>
<td>Poisson’s ratio</td>
<td>0.27</td>
</tr>
<tr>
<td>Density</td>
<td>2700 kg/m³</td>
</tr>
<tr>
<td>Cohesion (Intact Rock)</td>
<td>11 MPa</td>
</tr>
<tr>
<td>Friction Angle (Intact Rock)</td>
<td>45°</td>
</tr>
<tr>
<td>Dilation Angle (Intact Rock)</td>
<td>30°</td>
</tr>
<tr>
<td>Tensile Strength (Intact Rock)</td>
<td>5 MPa</td>
</tr>
<tr>
<td>Cohesion (weakness planes)</td>
<td>1 MPa</td>
</tr>
<tr>
<td>Friction Angle (weakness planes)</td>
<td>30°</td>
</tr>
<tr>
<td>Dilation Angle (weakness planes)</td>
<td>10°</td>
</tr>
<tr>
<td>Tensile Strength (weakness planes)</td>
<td>10 kPa</td>
</tr>
</tbody>
</table>

The Improved Unified Constitutive Model (IUCM) (Vakili, et. al. 2014) was trialled as an alternative to the UJ model. The IUCM is a constitutive model which has been developed to capture the complexities of a wide range of geotechnical applications, while maintaining simplicity in application for geotechnical practitioners. The approach uses a Mohr-Coulomb fit to the Hoek-Brown criterion for the peak strength determination, while the post-peak failure envelope is based on the properties of a completely crushed rockmass (friction angle between 35° and 55°). The transition between peak and residual strength, modulus softening, anisotropy and dilatancy are described in Vakili, et. al.

The key inputs for the IUCM are listed in Table 2. A comparison between the results obtained using the IUCM model vs the UJ model are given in Figures 7 and 8.

In-situ stress inputs were obtained from hollow inclusion (HI cell) stress measurements from the 17 level at George Fisher Mine and are as given in Table 3.
Figure 6  Stereonet depicting mapping data at the instrumentation sites, with the dominant defect orientation (bedding) as indicated

Table 2  IUCM material properties assigned to the Flac3D model

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Young’s Modulus</td>
<td>9.3 GPa</td>
</tr>
<tr>
<td>Poisson’s ratio</td>
<td>0.27</td>
</tr>
<tr>
<td>Density</td>
<td>2700 kg/m$^3$</td>
</tr>
<tr>
<td>Uniaxial Compressive Strength (North Site)</td>
<td>90 MPa</td>
</tr>
<tr>
<td>Uniaxial Compressive Strength (South Site)</td>
<td>110 MPa</td>
</tr>
<tr>
<td>Hoek-Brown $m_i$</td>
<td>7</td>
</tr>
<tr>
<td>Hoek-Brown Disturbance Factor</td>
<td>0</td>
</tr>
<tr>
<td>Anisotropy Factor</td>
<td>3.0</td>
</tr>
<tr>
<td>Joint $m_i$</td>
<td>3</td>
</tr>
</tbody>
</table>
Table 3  In situ stress inputs

<table>
<thead>
<tr>
<th>Principal stress component</th>
<th>Value</th>
<th>Dip</th>
<th>Azimuth</th>
</tr>
</thead>
<tbody>
<tr>
<td>Major</td>
<td>45 MPa</td>
<td>24°</td>
<td>62°</td>
</tr>
<tr>
<td>Intermediate</td>
<td>38 MPa</td>
<td>5°</td>
<td>154°</td>
</tr>
<tr>
<td>Minor</td>
<td>27 MPa</td>
<td>65°</td>
<td>256°</td>
</tr>
</tbody>
</table>

Resin anchored rockbolt and SMART cable reinforcement was modelled using the CABLE element in Flac3d, with properties as given in Table 4. Manufacturer’s specifications for reinforcement and published values for grout properties have been applied.

Table 4  Reinforcement element properties

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Modulus (cable)</td>
<td>195 GPa</td>
</tr>
<tr>
<td>Modulus (rockbolt)</td>
<td>205 GPa</td>
</tr>
<tr>
<td>Grout stiffness</td>
<td>670 MPa/m</td>
</tr>
<tr>
<td>Grout cohesion (cable)</td>
<td>1.5 MPa</td>
</tr>
<tr>
<td>Grout cohesion (rockbolt)</td>
<td>2 MPa</td>
</tr>
<tr>
<td>Cross-sectional area (cable)</td>
<td>1.43e-4 m²</td>
</tr>
<tr>
<td>Cross-sectional area (rockbolt)</td>
<td>3e-4 m²</td>
</tr>
<tr>
<td>Borehole perimeter (cable)</td>
<td>0.2 m</td>
</tr>
<tr>
<td>Borehole perimeter (rockbolt)</td>
<td>0.115 m</td>
</tr>
<tr>
<td>Tensile Strength (cable)</td>
<td>0.2 MN</td>
</tr>
<tr>
<td>Tensile Strength (rockbolt)</td>
<td>0.2 MN</td>
</tr>
</tbody>
</table>

Fibrecrete liner properties are given in Table 5. Published values and on-site quality control data were used to derive these properties. The thickness of the liner was determined by photogrammetry scans before and after Fibrecrete application.

Table 5  Shotcrete liner properties

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Elastic modulus</td>
<td>20 GPa</td>
</tr>
<tr>
<td>Poisson’s ratio</td>
<td>0.25</td>
</tr>
<tr>
<td>Cohesion</td>
<td>1 MPa</td>
</tr>
<tr>
<td>Friction angle</td>
<td>30°</td>
</tr>
<tr>
<td>Density</td>
<td>2 t/m³</td>
</tr>
<tr>
<td>Tensile strength</td>
<td>0.1 MPa</td>
</tr>
</tbody>
</table>
5 Key outputs and correlations with instrumentation data

Sidewall closure: correlations between modelled and actual sidewall closure are shown in Figures 7 and 8 for the north and south sites respectively. It can be seen that the modelled data from both the North and South sites (Figures 7 and 8) follow the measurement trends quite accurately until near the end of the monitoring period, where the model prediction indicates a sharp increase in the rate of closure compared to the actual measurements. Further data collection may clarify the reasons for this divergence.

Reinforcement load: modelled axial loads in the SMART cable elements are shown in Figures 9 and 10. The correlations between SMART cable data and model predictions are shown in Figures 11 to 14, for the hangingwall and footwall sidewalls. SMART cable data for the remaining cables in the array did not show appreciable loads and thus no credible correlations could be made with model data. According to the model results, the hangingwall shoulder cables at both sites should have both shown measurable load close to the collar, however for the reasons mentioned in the following paragraph, no load was measured in this zone.

Several features are noteworthy, firstly near the collar of the hole, the measured load from the SMART cables drops significantly (except in one case), whereas the modelled results show high loading at the collar. This is ascribed to imperfect grouting near the instrument collar due to the use of a wadding plug to contain the grout (collar to toe grouting method used). This would have resulted in the SMART instrument anchor/s closest to the collar being decoupled from the rockmass and thus unable to develop load.

Secondly, in both the hangingwall sidewall cables (modelled data), there is a load peak some 1.5 – 2.5m away from the sidewall. This load peak is also evident in the instrumentation data for the north site, but not for the south site (refer Figures 11 and 13). The reason for this load peak is ascribed to a localised, deeper seated band of shear strain which is evident in the model plot shown in Figure 15.

Overall, some correlations between modelled and field data can be identified, but they are probably not as good a match as would be required for rigorous design purposes at an operational level. When considering peak load predictions vs actual, a variance of between 10% (good) and 40% (poor) was achieved, which could be adequate depending on the level of the study (i.e. operational, feasibility, pre-feasibility, scoping).

Liner load: Shotcrete liner strain measurements are shown in Figures 16 and 17. It can be seen that all instruments are showing increasing negative strain, which implies increasing liner stress (i.e. compression of the liner skin). These strain measurements are plotted against predicted liner load from Flac3d and the results shown in Figure 18. A positive correlation between stress and strain, with a gradient (equivalent to the modulus of elasticity) of approximately 8.6GPa, is indicated.
Figure 7  Modelled vs measured sidewall closure for the north site. Solid lines represent the tape extensometer data, coloured squares are the modelled results for the equivalent time step (IUCM model). Solid black circles represent the UJ model results.

Figure 8  Modelled vs measured sidewall closure for the south site. Solid lines represent the tape extensometer data, coloured squares are the modelled results for the equivalent time step (IUCM model). Solid black circles represent the UJ model results.
A numerical modelling case study - correlation of ground support instrumentation data with a three dimensional inelastic model

G.J. Sweby, P.M. Dight and Y. Potvin

Figure 9  Modelled reinforcement loads (north site, viewed from the SW)

Figure 10 Modelled reinforcement loads (south site, viewed from the SW)
Figure 11  Correlation between modelled reinforcement load (red line) and measured load (blue diamonds), north site, hangingwall sidewall cable. Predicted peak load is 30% higher than actual

Figure 12  Correlation between modelled reinforcement load (red line) and measured load (blue diamonds), north site, footwall sidewall cable. Predicted peak load is 25% higher than actual
Figure 13  Correlation between modelled reinforcement load (red line) and measured load (blue diamonds), south site, hangingwall sidewall cable. Predicted peak load is 43% higher than actual.

Figure 14  Correlation between modelled reinforcement load (red line) and measured load (blue diamonds), south site, footwall sidewall cable. Predicted peak load is 10% higher than actual.
Figure 15  Load peak 2.5m into the sidewall cable, north site, corresponding to localised band of shear strain in the rockmass (highlighted)

Figure 16  Measured tangential strain in shotcrete liner, north site. All instruments are indicating compressional strain (-ve)
Figure 17 Measured tangential strain in shotcrete liner, south site. All instruments are indicating compressional strain (-ve)

Figure 18 Correlation between measured strain and modelled stress in the shotcrete liner (black squares – north site, red squares – south site)
6 Discussion and conclusions

Some degrees of correlation have been obtained between field instrument measurements and a Flac3d model. Realistic input data for rockmass properties, in-situ stress and support elements have been applied, with attention to detail in model initialisation and sequencing to ensure that a realistic loading path is followed.

The primary correlation parameter is the tunnel sidewall closure, using a proven, reliable method of measurement. These correlations are satisfactory and could be improved incrementally by further fine-tuning the input parameters.

The correlation between measured and predicted cable load is somewhat more erratic than the closure measurements, although trends are consistent. Considering peak loading only, the percentage difference ranged from 10% to 40% (good to poor) for the cables which showed measurable load.

The shotcrete liner measured strains vs model predicted loads gave an interesting correlation for a subset of the data, indicating a material modulus (for the shotcrete) of approximately 8.6GPa, which is believed to be within the range of underground application.

In regards to the applicability of numerical modelling for ground support design, this study has shown the following:

- This type of modelling requires a significant amount of time and expertise which is generally not available at mine site in practice;
- The correlations achieved is realistic for some parameters, but somewhat erratic for others;
- At the early stages of design, no verifiable data are available for correlation/calibration and thus results may not be reliable;
- The tool itself has the capability to reproduce the load and displacement experienced by ground support under controlled conditions such as this monitoring study;

In summary, it is difficult to conclude that this type of numerical modelling could be used for explicit design of ground support systems at mine sites given the complexities in setting up, running and calibrating the model. The sensitivity of the outputs to input data variability is also a major concern when using numerical models for design purposes (Sweby, et. al., 2014)

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Jennmar Australia
Dywidag-Systems International Pty Ltd
Fero Strata Australia
Golder Associates Pty Ltd
Geobrugg Australia Pty Ltd
Atlas Copco Australia Pty Limited

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A proposed workflow for slope stabilization projects. The case of an open pit mine in Norway
Kostas Botsialas, Titania AS, Norway

Abstract
Titlean open pit mine is placed in Sokndal Municipality in South - Western Norway. It is an open pit mine that produces ilmenite since 1960. The ilmenite orebody is considered as one of the largest in the world. The open pit has a length of about 2.8 km, while the depth today is at 240 meters. The width of the pit varies from 80 to 600 meters. Historically there have been numerous slope instability occurrences, particularly on the hangingwall slopes.

On September of 2014 in the aftermath of a rock fall, it was decided that an extended slope stabilization project against possible instabilities of minor/local scale was necessary. The duration of this project was from October 2014 until mid- April 2015. The total area that was investigated and stabilised was approximately 100.000 m².

The gained experience and the lessons learned during this project, led to the compilation of a workflow procedure which aims to find the optimum equilibrium between the 3 following conditions: effectiveness of supporting measures, effective time management and best possible budget management and allocation. This workflow attempts to establish a thread between the way that geological/geotechnical conditions are defining the type of mitigation measures, and the way that an effective time management plan can lead to the optimum budget planning and allocation.

1 Introduction

For every modern mining operation, the main aim of its pit design is to provide an optimal excavation configuration in the context of safety, ore recovery and financial return. Stock-holders and operators expect the slope design to establish walls that will be stable for the life of the open pit, which in some cases may extend beyond closure (Stacey 2010). As a result, any kind of instability should be manageable and if possible, it should not be developed to a failure. This framework applies at every scale of the walls, from the individual benches to the overall slopes.

On September of 2014, a failure took place on bench 80 at the hanging wall side. It was a rock fall that happened on a bench, which is used as an access road and where important utilities are installed. This incident triggered the initiation of a wall support and wall reinforcement project against local stability problems (bench scale failures). The duration of this project was from October 2014 until mid- April-2015. The stabilization project was commenced, by using different stabilization measures like: alteration of the slope geometry, installation of drainage, utilization of reinforcement techniques, use of external stabilization measures, or by using combinations of these methods.

Slope geometry modification was including scaling which was performed manually with pry-bars and picks. Air pillows (pneumatic pillows) were also used and in some cases spraying water on the rock slope face, at a rate of 1000lt/min, was proved quite effective. In the case of overhanging faces and protruding knobs, trim blasting was used. The latest was also used in cases where modification of the slope angle was necessary as there was a need to improve rock-fall trajectory and slope stability. The total area that was covered by utilizing various scaling techniques was approximately 100.000 m².

Reinforcement was comprised by installing more than 1800 two-speed resin bonded rock bolts. The aim was to increase the normal force friction and shear resistance along discontinuities and potential failure surfaces.
The rock bolts that were used are usually less than 6m long. There were also cases where bolts longer than 6m were used, reaching the length of 9m. The bolts working load is typically between 150 and 330kN and they are formed from high yield steel bars with diameters up to 32mm.

Regarding the external stabilization measures, safety net against rock falls was installed on a certain area of the pit, covering an area of 0.9 km$^2$. The wire of the safe-net is 3,7mm thick, and is fully galvanized. Finally in the cases where there was a need for reducing the water pressures within the slope, horizontal drains were used in conjunction with other design elements.

This project, that lasted 7 months, was proven quite challenging both for the geotechnical staff of the mine and the contractor that was implementing the stabilization measures. This project was an excellent opportunity for the mine staff to realize that a solid plan on how to perform a stabilization project of great extent, is needed. Therefore, a workflow was built based on the experience acquired. The main points of this workflow are the following:

- Establishing a framework agreement with the contractor that is going to implement the mitigation measures. Selecting the right partners.
- Preforming a prefeasibility study at the area of interest.
- Designing the stabilization measures.
- Communicating the design results to the contractor.
- Controlling the cost and quality of the work.

2 Establishing a framework agreement. Selecting the right partners

Selecting the right company that is going to implement the slope stability measures, is of great importance for the successful completion of any stabilization program. It is preferable that one contractor should cooperate with the geotechnical staff of the open pit in a long term basis. The main two reasons are as follows:

1. The interaction between the geotechnical staff and the employees of the contractor/supplier, is the main fundament for a reliable implementation of any stabilization measure. The establishment of a long term relationship, must be based on good communication between the two parties. The geotechnical engineer is responsible for the dimensioning and design of the stabilization measures while the staff of the supplier has the responsibility to implement them. Between these two situations, misunderstandings can take place, which may lead to an outcome of lower quality than the desired one.

2. The staff of the supplier must be familiar with the safety routines of the mine. Modern mine companies are having various training programs in order to train the contractors, before their entrance into the pit environment. Nevertheless, the full integration to the safety culture of each mine environment, comes after a certain time period is passed. At the same time, during the operations, many practical issues can arise where the involvement of mine staff, other than the geotechnical staff, like electricians, workshop technicians, may be required. The latest applies in the case that supplier’s equipment needs to be repaired. Thus quick access to the proper people and facilities is reducing the interruption time of the operations. A contractor who is familiar with the people and the procedures of the mine is far more effective and productive than one who has to use time and energy in order arrange issues of minor or significant importance.

As a result it is proposed that a framework agreement of 3 years must be established between the mine department and a company that implements stabilization measures.

The criteria for the selection of the most suitable contractor can be the following:
1. Experience in projects of similar nature. The contractor should be able to execute the following type of works:
   a. Installation of bolts of various types, diameters and lengths.
   b. Installation of rock fall nets.
   c. Installation of nets against ice falling.
   d. To perform manual scaling with the use of pry-bars and picks. Air pillows (pneumatic pillows) should also be able to be used and in some cases spraying water on the rock slope face, at a rate of 1000lt/min, is also desired.
   e. To perform scaling and bolting with the use of experienced climbers.

2. The equipment adequacy for projects of this kind. The capabilities of the lift regarding the height and the horizontal extent of the basket should cover the requirements associated with the bench heights of the open pit. For the current open pit, the bench heights are at 30 meters and the experience have shown that a lift with vertical reach between 48 and 53 meters, is the most appropriate for use.

3. The versatility of equipment against specific climatological conditions. It is quite often that winds up to 25m/sec, can last many hours during the day, or even for a period of two or three days.

4. The supplier’s ability to respond to states of emergency. For the mine department this limit has been set at 24hours.

3 Prefeasibility Study-Planning

An open pit mine is an organization where many activities are taking place at the same time, in different or adjacent pit domains. As a result, it is very important before the implementation of any stabilization project, to plan the project in close cooperation with the chief of operations of the open pit. The main purpose of this plan is to define at which pit areas, stabilization measures must take place and when. Thus, the activity plan should be able to classify the necessity of stabilization measures at different parts of the pit. The criteria for this classification should be the following:

1. At which areas of the open pit, activities will take place.
2. What type of activity will take place (driving, blasting, and installation of infrastructures etc.).
3. Which type of equipment will be used for each type of activity (size and type of the vehicles and/or machinery).
4. The amount of time that each type of activity requires to be performed.
5. If the area is monitored by any type of monitoring equipment (prisms, slope stability radars etc.).

This activity plan should be able to cover a time span of some months ahead. In the case of Titania this time period was set to 6 months.

3.1 Defining the safety levels. Zonation of the pit area

The next step in the prefeasibility study is to use the criteria mentioned above in order to compile a map where four different levels of the required safety are depicted:

- Level 1: This level applies to those areas of the pit, which are corresponding to the final wall, or to those parts of the wall where no pushbacks will be envisaged for a period of at least one year. Two of the following conditions should be met:
  - Benches that are used as access roads and their total width is not greater than 6 meters.
  - Medium or small size vehicles are using the access road every day.
Power supply units or any other kind of important infrastructures are lying on the foot of the bench slope.

The width of the bench is not allowing the construction of catchment ditch or a wall that can absorb the kinetic energy of possible rock falls.

- **Level 2**: This level applies to those areas of the pit, which are corresponding to the final wall, or to those parts of the wall where no pushbacks will be envisaged for a period of at least one year. Two of the following conditions should be met:
  
  - Benches that are used as access roads and their total width is between 6 and 20 meters or more. The road is used by a variety of vehicles from pick-ups to up to off-highway trucks daily.
  
  - Power supply units or any other kind of important infrastructures are not lying on the foot of the bench slope.
  
  - The width of the bench is allowing the construction of catchment ditch or a wall that can absorb the kinetic energy of possible rock falls.

- **Level 3**: This level applies to those parts of the wall where push backs will be envisaged in a period less than one year. Two of the following conditions should be met:
  
  - Benches which are used as access roads, and their total width is less than 6 meters. Medium or small size vehicles are using the access road every day.
  
  - The width of the bench is not allowing the construction of catchment ditch or a wall that can absorb the kinetic energy of possible rock falls.

- **Level 4**: This level applies to those parts of the wall where push backs will be envisaged in a period less than one year. Two of the following conditions should be met:
  
  - Benches which are used as access roads, and their total width is between 6 and 20 meters or more. The road is used by a variety of vehicles from pick-ups to up to off-highway trucks daily.
  
  - The width of the bench is allowing the construction of catchment ditch or a wall that can absorb the kinetic energy of possible rock falls.

- **Level 5**: No activity will take place in the forthcoming period of 6 months. This area will not be assessed for implementation of stabilization measures.

The zonation of the area of interest in different levels of safety, is subsequently leading to the classification of the bench slopes under investigation in 5 classes in terms of the desired factor of safety (table 1, Figures 1 and 2):

<table>
<thead>
<tr>
<th>Safety level/class</th>
<th>Desired Factor of Safety</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Long term stability: F ≥1.5</td>
</tr>
<tr>
<td>2</td>
<td>Long term stability: 1.3≤F ≤1.5</td>
</tr>
<tr>
<td>3</td>
<td>Short Term stability: F≈1.3</td>
</tr>
<tr>
<td>4</td>
<td>Short term stability: 1.1≤F ≤1.3</td>
</tr>
<tr>
<td>5</td>
<td>No stabilization measures required.</td>
</tr>
</tbody>
</table>
Figure 1  Zonation of two benches slope, based on anticipated safety level. Zones 1, 2 and 5 are depicted here

Figure 2  Zonation of two benches slope, based on anticipated safety level. Zones 3, 4 and 5 are depicted here
3.2 Prioritizing the areas for the implementation of stabilization measures

This is the last step of the planning procedure. The order of the areas that are going to be stabilized is strongly depended, on the operation plans of the mine. The quality and quantity of the delivered job is closely related to eventual disruptions caused by other operations in areas close to the one where the implementation of stabilization measures is taking place. Therefore, it is important for the contractor to have enough space and time to deploy its equipment, and at the same time, it is not causing any disruptions to the operation of the mine itself.

4 Design of the stabilization measures

4.1 Acquiring a solid overview of the area of interest

The overview of the area of interest should done at least in two scales of observation:

The first scale is the “macro” scale, which means that the area under investigation should observed by such a distance, that someone could have a holistic overview of the situation. It is important to understand the type and mechanism of possible failures, on the different parts of the slope. In addition, it is crucial to define the interaction between the different rock blocks. Some blocks must be addressed as standalone cases while some others are a part of a system of two or more blocks. This discrimination can be used as a guide regarding the type of measures to be implemented. For example if there is block with a volume of 2m³, which is standing isolated (it is not interacting with other blocks) on a possible failure plane of 45° with low residual friction angle (<28°), the most preferable measure is to remove it during scaling. On the other hand if the same block is part of a system of 2 or 3 blocks which are standing on the same possible sliding plane, and they are applying lateral forces between them, then it can be quite preferable to bolt the block, as possible removal can create more severe stability issues of local scale.

At the macro scale the utilization of a panoramic image of the area of interest can be a very useful tool. On this image, the blocks can be sketched and delineated. Then with the use of a laser range finder, someone can measure the basic dimensions of each block. In addition, one can measure the angle of a probable failure plane and the slope angle as well. These three numbers are adequate to have a rapid estimation of the stability condition according to the general conditions for failure plane for example.

In many cases, digital photogrammetry methods were used to produce 3D orthorectified digital images of the sections under investigation. The central component of the system is a software (Sirovision) that enables someone to create a 3D image from two suitable digital images. The 3D image is an accurate model of the surface of the rock mass integrated with a visual image of the rock mass, registered with the 3D spatial data. This property allows the user to map planar and linear features on a 3D image of a rock mass and to measure attributes of these features. The attributes measured in the first wave of mapping are dip, dip direction and persistence (Figure 3). Someone can then proceed in the definition of orientation sets, their average spacing and persistence, along with min and max values. This data can then be easily exported in a variety of formats to be used with other discontinuity analysis tools such as DIPS or Georient.

Subsequently, a closer examination of the rock mass properties will be quite helpful. This will allow a thorough view of the conditions of the discontinuity planes that can induce an instability. Parameters like the roughness and the type of the infilling material or the joint conditional strength can be measured and registered for use during the dimensioning procedure. In addition, phenomena like, the effect of blasting on the rock, the ground water flow along the fractures, the gradual (or not) change of rock mass properties around the fracture/fault zones, the weathering and ice/thaw effect can observed closely.

Additionally bolts that were installed in the previous bolting campaigns can be evaluated in terms of their efficacy. In cases that the quality of the bolt is affected by corrosion, new bolts must be installed. In the current open pit, bolts or dowels that were installed after 2012 are resistant to corrosion. The majority of the bolts or dowels that were installed before 2012, were not corrosive resistant.
4.2 Zonation of the area of interest according to the size of the rock blocks

A holistic overview of the area under investigation, is leading to the next step, which is the zonation of the area, according to the size/weight of the blocks, which are the majority at different parts of the wall under investigation and are considered as unstable. This type of zonation is a quite important step, as it can lead to a preliminary delineation of the type of the stabilization measures that have to be implemented, in each of the delineated zones.

The size of the rock blocks in terms of volume is cubic meters. For the calculation of the volume, the shape of potentially unstable blocks is “rounded” to the basic shapes of pyramidal, cubical shaped wedges. For the measurement of the basic dimensions, the use of a laser range finder, was proven quite useful and accurate.

By using the experience of previous stabilization programs, the classification that is used in the current open pit mine is shown in table 2.

Table 2 Classification of the potentially unstable rock blocks of the area of interest, according to their size

<table>
<thead>
<tr>
<th>Class of rock blocks size</th>
<th>Range of the rock blocks size</th>
</tr>
</thead>
<tbody>
<tr>
<td>5</td>
<td>V&lt; 1m³</td>
</tr>
<tr>
<td>4</td>
<td>1m³&lt;V&lt;5m³</td>
</tr>
<tr>
<td>3</td>
<td>5m³&lt;V&lt;10m³</td>
</tr>
<tr>
<td>2</td>
<td>10m³&lt;V&lt;20m³</td>
</tr>
<tr>
<td>1</td>
<td>V&gt;20m³</td>
</tr>
</tbody>
</table>
Based on the given classification, the part of the slope walls, which is under investigation, should be divided into zones according to the block size that numerically prevails in each zone. These zones should be drawn on a map, or a panoramic image. Thus, those parts of the wall where the majority of the blocks are belonging to class 1, are characterized as zone 1. By following this procedure, the whole slope is divided in 5 classes/zones. An example of the zonation based on the block size is shown in the figure below (Figure 4):

![Figure 4](image)

**Figure 4** Zonation of two benches slope, based on the size of the potentially unstable rock blocks

### 4.3 Dimensioning the stabilization measures

The dimensioning of the stabilization measures is that intermediate step, where the observations made in previous steps, must be converted into numbers. This step applies mainly in the case of implementation of reinforcement methods. In the case of Tellnes open pit, the rock blocks that should be reinforced were potential rock falls, plane or wedge failures.

The stability analysis that was performed for the potentially unstable blocks/areas was based on the traditional deterministic principals, with calculation of one single of factor of safety for each of the cases under investigation. A typical deterministic stability analysis includes the following:

1. The identification of potential instabilities
2. The quantification of input parameters and calculation of stability
3. The dimensioning of reinforcement.

#### 4.3.1 Defining the potential unstable areas

For the definition of the potential stability problem, the main question that has to be answered is which blocks or parts of the slope can be characterized as unstable or potentially unstable. A monitoring system, like a slope stability radar, gives adequate information about a certain area, and can be used as a guide. Areas which are not monitored are evaluated with the use of traditional evaluation methods.

For potential plane and wedge failures, the geometry of the rock blocks was used for analysing the basic mechanics of sliding in both cases (Figure 5). The general conditions of plane and wedge failures are very well portrayed by Wyllie and Mah (2010).
4.3.2 Quantification of input parameters and calculation of stability. Dimensioning the reinforcement

At this step, it was decided that it is important to make a discrimination on which type of methods should be used for the quantification of input parameters and for the calculation of stability. The background of this decision was the fact that 95% of the rock blocks were lying into the first 3 classes of block size (table 2). Class 4 was representing 3% of the cases while 2% was lying in the 5th size class. This means that 95% of the blocks were having a volume less than 20m$^3$. Many of those blocks were removed during the implementation of slope modification techniques (scaling, water spraying etc.), but still, their total number was quite high, and therefore the utilization of conventional methods in quantifying the input parameters and calculating the stability was proven time consuming. The time that should be used in favour of accuracy, was against the progress of the whole project.

The general procedure was, that first the dimensioning was done on a certain sector and then the results were communicated to the operators for a time span of one working day ahead. Then dimensioning was performed again for the subsequent sector, and the results were passed to the operators as soon the previous areas was finished by them.

Into this framework, it was not possible to produce the amount of results that were needed in order to establish a smooth and uninterrupted “production line”, by following conventional techniques in quantifying the input parameters and in calculating the factor of safety in the most accurate way.

Therefore, it was decided to use an unconventional approach. This approach was introduced by Nilsen and Hagen (1990), and has been used numerous times in the past in Titania open pit, with very good results in terms of safety level that was reached.

According, to this method the factor of safety is based on the typical equation of stabilizing and driving forces:

$$ F = \frac{\text{Stabilizing Forces}}{\text{Driving Forces}} = \frac{R}{S} \quad (1) $$

Since a rock block or an area are fulfilling the criteria described above, or signs of instability have been registered by a monitoring system or by a visual observation, then the current factor of safety at that current moment is considered as 1. In order to reach the desired factor of safety (1.3 in the case of level 3 safety class for example), the total capacity per meter that the installed bolts should have, in order to increase the normal force friction and shear resistance along discontinuities and potential failure surfaces should be:

$$ P(\text{tot}) = 0.3 \times S \quad (2) $$
Where: $P(tot)$ is total bolt capacity per meter.

As main driving force is considered that component of the total block weight ($W$) which is parallel to the potential failure plane. If the sliding plane has a dip angle $\psi_p$ then the previous equation 2 can be written as:

$$P(tot) = 0.3 \times W \sin \psi_p$$  \hspace{1cm} (3)

Where:
- $W$ is the weight of the block and
- $\psi_p$ is the angle of the possible sliding plane.

Equation 3 gives the total capacity of the bolts per meter that must be applied in order to reach the desired factor of safety.

It is obvious that this method overestimates the stabilization measures, but at the same time it is a convenient way for a rapid calculation of the total bolt capacity that must be applied. The water pressure is not measured but at the same time the friction angle of the sliding plane is considered as zero. The implementation of this method in practice was proven quite effective and easy to use.

For those cases that are lying into the 2nd and 1st volume class (i.e: volume > 10m$^3$), a conventional deterministic analysis was used. In this type of analysis the joint friction and the water pressures were counted in the final calculation of the factor of safety.

For the joint friction, the Barton Bandis criterion (1990) was used in the majority of the cases:

$$\tau = \sigma_n \star \left[ JRC \star \log \left( \frac{JCS}{\sigma_n} \right) + \varphi_b \right]$$  \hspace{1cm} (4)

Where:
- $JRC$: is joint roughness coefficient ,
- $JCS$: is joint compressive strength,
- $\varphi_b$: is basic friction angle.

The $JRC$ and $JCS$ were calculated by taking into account the scale effects as described by Barton and Bandis.

The water pressure, as Hoek and Bray (1991) suggests, has a triangular distribution during heavy rainfall. This distribution represents free infiltration of water from the top of the slope into the rock mass. The water pressure reaches its maximum value corresponding to the hydrostatic one at a height equal to 50% of the slope height. Therefore the maximum water pressure was calculated as:

$$U_{\text{max}} = \gamma_w \star H^2 / 4 \star \sin \psi_p$$  \hspace{1cm} (5)

According to Nilsen (2000), due to the inhomogeneous and discontinuous character of rock masses, this idealized triangular distribution seldom corresponds perfectly with the real situation during heavy rainfall. However, in the absence of better alternatives, this is the configuration most commonly seen in literature. Nilsen (2000) suggests that the triangular distribution often exaggerates the resultant pressure, because joints and cracks frequently provide a degree of drainage towards the slope face. This view was taken account into the calculation of the factor of safety.

The calculation of the factor of safety and the dimensioning of the reinforcement were performed by the use of the software suites RocPlane and Wedge, which are developed and provided by Rocksciences.

5 Communicating the dimensioning results to the contractor

The dimensioning is a procedure that is compiled by the geotechnical engineer of the site and is considered as the first node of the stabilization project. The implementation of the results is the other end of the whole procedure. Between these two nodes, the communication of the results is that step of the procedure
which is the most crucial one, so that the deviation between the design and the implementation is the smallest possible. At the same time the adequate communication of the dimensioning results, is not an one way procedure from the engineer to the operator. During the project, there have been cases where the feedback from the operators, led to the adjustment of the proposed measures, so that their implementation was applicable in the most optimum way.

In general, there are three prerequisites, so that the communication of the results can be considered as effective:

1. Clear and explicit delineation of the type of stabilization measures and of the areas where each type of measures will be implemented. Especially, in the case of reinforcements methods, the pattern, the type and the length of bolting should be clarified with the contractor before the installation of the bolts. Pictures, where the position of the bolts can be used, and in some cases the use of the lift should be also considered. In the case that the lift will be used the engineer along with the operator, can use sprays of different colours for different type of bolts.

2. The time span of the work that should be done ahead should not cover more than one working day. It was seen that in many cases, if the plans were covering more than one working day, misunderstandings took place, that led to deviations from the designed measures. This fact, subsequently led to delays, as quite a lot of time needed to check the quality of the job that has been done and to revise the plans for the correction of the errors made.

3. At the end of each working day, or before the beginning of the next one, a meeting should take place between the geotechnical engineer and the operators. During this meeting, the operator, should give an overview of the measures that were implemented to the geotechnical engineer. The challenges met during the stabilization procedure should be reported to the engineer. The latest should assess the information provided, and after discussion with the operators, small adjustments should be made for the subsequent steps of the project.

6 Implementation of the reinforcement

For the installation of the bolts the following nine steps were followed (Wyllie and Mah, 2010; Littlejohn and Bruce, 1977; FHWA, 1982; BSI, 1989; Xanthakos, 1991; Wyllie,1999):

- **Step 1: Drilling**—At this stage, it is very crucial to establish easy access for the equipment and the operators to the area of interest.

- **Step 2: Bolt materials and dimensions.** Generally the following materials were used:

<table>
<thead>
<tr>
<th>Type</th>
<th>Diameter (mm)</th>
<th>Corrosion protection</th>
<th>Bond Type</th>
<th>Rupture Load (tonnes)</th>
<th>Yield Load (tonnes)</th>
</tr>
</thead>
<tbody>
<tr>
<td>M16 0.8 and 1.5 m.</td>
<td>16</td>
<td>Zinc Coating</td>
<td>Two speed resin grout</td>
<td>8</td>
<td>6</td>
</tr>
<tr>
<td>M20 1.5 m, 2.4m, 3.0m, 4.0m and 6.0m</td>
<td>20</td>
<td>Zinc Coating</td>
<td>Two speed resin grout</td>
<td>15</td>
<td>12</td>
</tr>
<tr>
<td>M24 4.0 m and 6.0m</td>
<td>25</td>
<td>Zinc Coating</td>
<td>Two speed resin grout</td>
<td>24</td>
<td>20</td>
</tr>
<tr>
<td>Ø 32 9m and 12m</td>
<td>32</td>
<td>Zinc Coating</td>
<td>Cement grouted</td>
<td>40</td>
<td>35</td>
</tr>
</tbody>
</table>

In most of the cases the diameter of 25mm was found the most convenient for usage. The reason behind this selection was the fact that a 25mm bolt has 60% bigger capacity in relation to a 20mm bolt. At the same time the price of a 25mm bolt is less than 5% higher than the one of a 20mm bolt. In addition, less bolts will have to be used in order to reach the desired capacity. Thus the total cost for the stabilization of a block is much lower in the case of 25 mm bolts.

- **Step 3: Corrosion**—The environment of the site is highly corrosive. The high concentration of phosphorus and sulfide compounds in the atmosphere along with the high humidity and the
proximity to the sea are creating a quite corrosive environment. Thus all the material were protected against corrosion, with the use of zinc coating.

- **Step 4: Bond type**—For the diameters between M16 and M20 two speed resin grout was used while for the diameter of 32 mm cement grout was used to secure the distal end of the anchor in the hole.

- **Step 5: Bond length**—The required bond length was calculated with the use of the following formula Wyllie and Mah (2010):

\[
 l_b = \frac{T}{\pi d_h \tau_a} 
\]

Where:
- \( l_b \) is the design bond length
- \( T \) is the design tension force
- \( d_h \) is the hole diameter
- \( \tau_a \) is estimated from the uniaxial compressive strength of the rock in the anchor zone according to the following relationship Littlejohn and Bruce (1977):

\[
 \tau_a = \frac{\sigma_i}{30} 
\]

- **Step 6: Total anchor length**—The total anchor length, which is the sum of the bond length and free stressing length. “The free stressing length should extend from the rock surface to the top of the bond zone, with the top of the bond zone being below the potential sliding plane” Wyllie and Mah (2010).

- **Step 7: Anchor pattern**—The layout anchor pattern was based on the general principal that the bolts should approximately evenly spaced on the face. The following equation was used for the calculation of the vertical spacing between the bolts Wyllie and Mah (2010):

\[
 S_v = \frac{B n}{T} 
\]

Where:
- \( S_v \) is the vertical spacing,
- \( B \) is the design tension force in each bolt,
- \( T \) is the support force per unit length of slope, then the required,
- \( n \) is the amount of horizontal rows of the bolts.

- **Step 8: Waterproofing drill holes**—After the drilling and before the installation it was checked that there were no discontinuities in the bond zone into which grout could leak. In the case that leak was founded the hole was sealed by grouting and re-drilling was performed.

- **Step 9: Testing**—A testing procedure was set up to verify that the bonded length could sustain the design load. For each diameter at least two tests were performed during the realization of the project.

## 7 Cost checking and follow up

At the end of each working week, the total amount of measures that were implemented should be registered. The cost associated with each type of measure should be calculated and invoiced. The comparison between the final cost and the one that was initially calculated, must take place. If the deviation between the two values is more than a certain threshold in favour of the actual cost, then a meeting should take place between the contractor and the geotechnical engineer. In this meeting the
reasons for this deviation should be analysed. The conclusions of this meeting can lead either to revision of the dimensioning or to adjustment of the installation procedure.

8 Conclusion

In this publication an effort was made to establish a solid workflow, under which the design and implementation of stabilization measures can be performed in the most effective way. A thread was established between the geological conditions, the implementation of the stabilization measures, the design of the measures and the prefeasibility study and planning. The established workflow is including the following steps and sub-steps:

1. Establishing a framework agreement with the contractor that is going to implement the mitigation measures
2. Performing a prefeasibility study for the area of interest in which zonation of the pit area according to anticipated factor of safety at each area zone, is compiled. Subsequently the areas where the implementation of stabilization measures will take place, are prioritised according to the operational needs
3. Designing the stabilization measures. The following sub steps can be followed:
   3.a Acquiring a solid overview of the area of interest
   3.b Zonation of the area of interest according to the size of the rock blocks
   3.c Dimensioning the stabilization measures for each zone
7. Communicating the design results to the contractor by giving a clear and explicit delineation of the type of stabilization measures and of the areas where each type of measures will be implemented
8. Implementation of the reinforcement
9. Controlling the cost and quality of the work. At the end of each working week, the total amount of measures that were implemented should be registered. The cost associated with each type of measure should be calculated and invoiced. Comparison between the final cost with the one that was initially calculated must take place. If a deviation is taking place over a certain threshold, in favour of the final cost, then the appropriate adjustments should be done in order to avoid recurrence of this divergence.

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References

A risk-based approach to ground support design


*SRK Consulting, South Africa. *Australian Centre for Geomechanics, Australia

Abstract

The objective of installing support is to mitigate the risk of injuries and costs associated with rehabilitation of excavation damage and associated production downtime. It follows that the greater the likelihood of stress damage or rockfalls, due to the inherent rock mass conditions, the more work is required from the support system. In addition to this, the exposure of personnel in these excavations and the quantity of production serviced by the excavation will significantly influence the support requirements.

While support systems have evolved over many years to cater for these demands and experienced rock engineering practitioners generally apply appropriate engineering judgement, most design methods are based on a simple factor of safety and do not cater for a proper risk evaluation. In nature, rock mass characteristics are variable and considerable effort is required to understand the variability, particularly when the geology is complex. Stress fields are often unknown or are simply inferred from one or two stress measurements.

This paper describes a risk-based approach to support design, which takes the variability of rock mass conditions into consideration. A statistical block stability method is used to estimate the probability of occurrence of rockfalls of different sizes in a given length of tunnel. Elastic and elasto-plastic numerical methods are used to estimate the probability of the depth of failure exceeding a prescribed serviceability criteria. The results of these analyses are used to estimate the expected cost of excavation damage, based on the rehabilitation cost and associated production downtime.

1 Introduction

The need to change from deterministic, factor of safety design methods to probabilistic methods and to appropriately address uncertainty has been discussed by many authors (Harr 1996; Steffen et al. 2008; Tapia et al. 2007; Contreras 2015; Wesseloo & Read 2009). Many powerful methods for probabilistic evaluation of geotechnical stability have been developed over the years but despite this, is not widely applied in underground mining geotechnical applications. This is partly due to the additional effort required for probabilistic analyses and a lack of tools to make such analysis easily achievable on a mine site. Another stumbling block seems to be the lack of prescribed “acceptable probabilities of failure” which serves as a design acceptance criteria.

It is our conviction, though, that a design acceptance criteria based on probability of failure has little meaning within the mining context as the important factor that needs to be managed is not failure but risk. In mining, failure with no adverse consequence is preferable to unnecessary stability at a high cost.

We therefore argue for a risk-based design approach as opposed to a probabilistic design approach. A risk-based design approach is simply design approach for which the acceptance criteria is defined in terms of risk and therefore includes probabilistic assessment and risk evaluation components. Several authors have proposed methods of risk evaluation (Contreras 2015; Steffen et al. 2008; Tapia et al. 2007; Stacey 2007; Abdellah et al. 2014; Joughin et al. 2012a; Wesseloo & Read 2009; Brown 2012).

The objective of this paper is to describe a relatively simple process of carrying out a risk-based support design. This process incorporates block stability and stress damage analyses and provides a means of
estimating the probability of occurrence and potential costs associated with the damage. A method of communicating the risk is suggested. The estimation of potential injuries for safety risk is important, particularly from an ethical perspective, but is not discussed in this paper. A method of evaluating the likelihood of injuries is described in (Joughin et al. 2012a; Joughin et al. 2012b).

2 Risk-based design of support

The risk-based support design process is described using the simple examples of tunnels supported with bolts and mesh (Figure 1). If the support system is appropriate for the prevailing stress state and ground conditions, then it is expected that the tunnel will function effectively and there will be no disruptions. However under certain circumstances the tunnel may experience damage.

Two possible modes of failure, namely joint bounded rockfalls and stress damage, are illustrated in Figure 2 are considered in this study. If a rockfall occurs, some effort will be required to remediate the rockfall, which will depend on the size of the rockfall. The support system will be able to cater for a limited amount of stress damage. When the deformation exceeds the capacity of the support, or the tunnel converges to such an extent that mobile equipment cannot pass, it will be necessary to rehabilitate the tunnel. The cost of removing failed rock and re-supporting the tunnel will be a function of the extent of damage or size of the rockfall. However, the loss of revenue during the time that the tunnel is being remediated is often far greater than the cost of remediation. In addition to these losses, equipment may be damaged if the timing and location of rockfalls is coincident with equipment. The potential impact on revenue will depend on the location of damage and the purpose of the excavation. Injuries may also occur if the timing and location of rockfalls is coincident personnel. Injuries and fatalities may have severe financial implication and should be considered but the ethical aspects surrounding injury and fatality need to be evaluated separately (Joughin et al. 2012a).

It should be noted that with a risk-based design approach the reduction of risk to personnel does not necessarily relate to a reduction in the probability of failure. Risk mitigation strategies form part of the design and accepting a higher probability of failure may result in a more economical design if the risk to personnel is mitigated, for example by preventing exposure of personnel by replacing them with remote controlled or automated vehicles.

Figure 1 Risk-based support design example – a tunnel supported with bolts and mesh
A rigorous risk-based design process is required to analyse and evaluate the risk as outlined in Figure 3. Typical input data for the analyses are listed in the oval shapes in the top central portion of the figure. These inputs are variable and may not be well quantified or understood. Probabilistic models of the input variables needs to be derived. As discussed by (Wesseloo 2016) the probabilistic model of the input variables are more than simply a frequency distribution of data as it also contains engineering knowledge of the problem constraints.
Figure 4 shows a typical sub-level for a longhole stoping operation. The examples in this paper will be based on the access ramp and primary stope drives.

The likelihood of rockfalls and stress damage can be analysed using block stability analyses and numerical modelling. The results of these analyses are probability distributions of rockfall size and stress damage. To enable the evaluation of the risk a model is needed to be developed to estimate potential losses associated with damage.

The final results are presented in terms of probability versus potential loss. These can be interpreted by mine planners in terms of a standard corporate risk matrix.

![Figure 4: Example mining layout for longhole stoping](image)

### 2.1 Rockfall (probabilistic block stability analysis)

This probabilistic block stability analysis in this study was performed using the JBlock software (Esterhuizen 2003). The application of this type of probabilistic block stability analysis in a risk-based design method was originally developed for support design in South African narrow tabular stopes (Joughin et al. 2012a; Joughin et al. 2012b) and is further adapted for application in tunnels.

#### 2.1.1 Input data

JBlock requires probability distributions for the orientation and spacing of each joint set. The first in developing these probability distributions is of course to collate joint data from oriented core (Figure 5) or scanline mapping techniques. Joints should be sorted into different sets based on their orientation using stereographic techniques. The true spacing of each joint set must be measured. The field data forms the bases for the development of the probability distributions.

JBlock uses the Mohr-Coulomb criterion for joint strength. It is recommended that the joint friction angles are determined by observing and assessing the joint conditions on site using the Q system (Barton 2002) with the following assessment of frictions angle:

\[
\tan^{-1} \phi = \frac{Jr}{Ja}
\]

where \( Jr \) and \( Ja \) are the joint roughness and joint alteration in the Q system.
Estimating the friction angle in this manner allows large amounts of data to be collected quickly and easily. Also, the friction angles of filled joints, which are most critical, can be estimated reliably (Barton 2002; Barton & Bandis 1990). The resulting distribution is somewhat irregular (Figure 5), due to the standard values of $J_r$ and $Ja$. A current limitation in JBlock is that it only has the truncated normal distribution available to model the probability distribution of the joint friction angle. This is a limitation that should be addressed, but for the purpose of this paper is not critical. The truncated normal distribution can be used to model thick tailed skew distribution representative of the expected distribution of friction angle in the field Figure 5.

![Figure 5 Processing of joint input data for rockfall analyses.](image)

2.1.2 Analyses

The next step is to generate a large set of three dimensional blocks. The joint set parameters and the orientation of the tunnel roof or crown are required to generate the blocks. A simple discrete fracture network (DFN) method is used to generate individual blocks, by randomly sampling the joint input data (Figure 6). For a robust analysis it is necessary to generate about 100 000 blocks. JBlock also cumulates the surface area of the base of each block and the surface area of DFNs, which do not result in the formation of blocks. In this way, JBlock keeps track of the simulated roof area represented by the set of blocks generated.

The tunnel outline and support pattern is simulated in JBlock (Figure 7). Simple models are used to represent bonded or end anchored bolts. Mesh is very simply modelled as the retaining force per square metre. This is another limitation of the software which is applied here outside of the original intended use, namely, tabular stope support assessment. This limitation is not important for the purpose of this paper,
namely to illustrate the risk design process, rather than to discuss the state of the art on block stability analysis.

The rockfall simulation involves the random placement of each of the blocks within the outline of the excavation. JBlock checks the position of support units relative to the block and performs a limit equilibrium block stability analysis. The bond length of each bolt in each block is directly simulated. A rotational mode of failure is also incorporated in the model. Figure 8 illustrates the rockfall simulation. The blue block traces represent stable blocks, green block traces represent blocks that out of range (excluded) and red block traces represent failed blocks or rockfalls. The block volume, base surface area, height, mode of failure and support information are reported for each block. During the simulation, JBlock cumulates the equivalent tunnel roof area analysed.

Figure 6  Discrete fracture network (DFN) to generate blocks

Figure 7  Tunnel outline and support in JBlock

Figure 8  Rockfall simulation in JBlock
2.1.3 Rockfall volume frequency distributions

The results are presented as a rockfall volume frequency distribution, normalised per 100 m length of tunnel in Figure 9. Note the frequency distribution is presented in three parts (0.1 to 1.0; 1 to 10 and 10 to 100), each with different bins sizes. These data can be used to estimate the likelihood of losses due to rockfalls (section 2.3).

![Rockfall volume frequency distribution](image)

Figure 9  Rockfall volume frequency distribution

2.2 Stress damage (numerical analysis)

The purpose of the stress damage analysis is to evaluate the likelihood and extent of tunnel rehabilitation that may be required. This will depend on the prevailing stress field, changes in stress, tunnel geometry, the rock mass characteristics and installed support.

2.2.1 Input data

The first step in the process is to characterise the variability of the rock mass strength. For the purposes of this example, the Hoek-Brown strength criterion was used (Hoek et al. 2002). The Geological Strength Index (GSI) (Marinos & Hoek 2000; Marinos et al. 2005) can be estimated from geotechnical core logging either directly or by converting from the raw inputs of other rock mass classification systems (Hoek et al. 2013). The process of determining the variability of GSI is illustrated in Figure 10. Logged GSI data should be captured in a geotechnical model.

The GSI data needs to compositied over regular intervals representative of the problem dimension. This is necessary to build a probabilistic model of GSI that captures the variance of the GSI parameter at the scale of the problem. In the case of a 5 m wide tunnel, the interval length should be in the order of 5 m to 10 m. GSI data should then be selected for the area of interest. A design probability distribution can then be derived from this frequency distribution of these data sets.
Intact rock strength should be estimated from laboratory tests (uniaxial compressive strength (UCS), triaxial compressive strength (TCS), with at least three different confinements, and Brazilian (BTS) and/or direct tensile tests (DTS). The Hoek-Brown strength envelope for intact rock should be fitted to the data as illustrated in Figure 11. In this study we assumed m, to be constant. Assuming a constant mi allows one to calculate the equivalent UCS value for each of the datapoints and obtain a distribution capturing the variance of the strength envelope.
2.2.2 Analyses

There are several numerical methods for estimating the extent of stress damage, ranging from simple empirical closed form solutions (Martin et al. 1999; Cai & Kaiser 2014) to complex three dimensional elasto-plastic numerical models. In this example an elastic modelling approach and a two dimensional elasto-plastic approach was used.

Ideally, a maximum deformation should be used as a serviceability limit state criterion. However, these continuum models are more reliable at estimating the depth of failure due to bulking (Kaiser & Cai 2013). As a result the depth of failure was chosen in this example as a serviceability criterion. The limiting depth of failure should then be selected on the basis of experience and limitations of the support system being considered.

Figure 11 Determining the variability of intact rock strength
The elastic analysis (Figure 12) was performed using Map3D and the probabilistic analysis of depth of yielding was performed using an mXrap (Harris & Wesseloo 2015) app which is discussed in more detail by (Wesseloo 2016).

The elasto-plastic modelling approach (Error! Reference source not found.) requires longer solution times and it is generally not practical to run many models and therefore a Monte-Carlo (MC) (Kroese & Rubinstein 2012) approach is not feasible to determine the probability of excessive damage. Three methods, namely the Point Estimate Method (PEM) (Rosenblueth 1975; Rosenblueth 1981; Harr 1989; Christian & Baecher 1999; Valley et al. 2010), response surface method (RSM) (Bradley 2007; Langford & Diederichs 2015; Lü & Low 2011), and the response influence factor (RIF) (Tapia et al. 2007; Wesseloo & Read 2009; Chiwaye & Stacey 2010; Steffen et al. 2008) are commonly used to perform probabilistic analysis in these cases where computational efficiency is critical.

In this study, elasto-plastic analyses were performed with the Itasca Code FLAC3D and the RIF method was implemented using the FISH language. Parametric models were set up to vary the tunnel geometry to model overbreak (as a result of blasting) and change the UCS and GSI or any other parameter. Figure 13 shows the results for different trials necessary for the RIF method.

Based on these analysis the probability distribution of the depth of yielding is obtained. The results for a section of the access ramp is shown in Figure 14. In this paper a serviceability criterion of 2.2 m depth of failure is considered and the probability of failure can be read off the graph (5%). This result represents the damage potential in an access ramp (see Figure 4), which is not significantly influenced by the stresses induced by stoping. For a primary stope drive in the same rock mass, the probability of exceeding a depth of failure of 2.2 m was 20%, due to the direct influence of stoping.
Figure 13  Model results (RIF) (Blue zones indicate tensile failure, red zones indicate shear failure)

Figure 14  Model output probability distribution (access ramp example)
2.2.3 Probability distributions of damaged tunnel length

It is necessary to determine the probability of occurrence of damage affecting a given length of tunnel for numerical stress analysis. The probability of failure in a numerical analysis, refers to the probability of exceeding a given damage/deformation criteria within a short length of a generic tunnel (p). This length ideally references to the composite interval length used for determining the variability of the rock mass characterization data.

In practice, the damage may occur over a greater length of the tunnel. This can be analysed, by considering that the tunnel comprises several discrete segments. The total number of segments (n) is the length of tunnel under consideration divided by the segment length (Figure 15). During the period under consideration, one or more tunnel segments may experience damage, which may not necessarily be contiguous. The overall length of damaged tunnel is the number of tunnel segments multiplied by the segment length.

It is reasonable to assume that the same p applies to each discrete tunnel segment. The probability of one or more tunnel segments being damaged (Pr) can be estimated using the binominal distribution:

\[
Pr = \frac{n!}{k!(n-k)!} p^k
\]

Where k is the number of tunnel segments affected.

It is then possible to obtain a probability distribution for damaged tunnel length. Figure 16 shows the probably distributions for damaged tunnel length for an access ramp (p = 5%) and for primary stope drives (p = 20%). It is apparent that damage is expected to be more significant in the primary stope drives. This can then be applied in the estimation of the losses due to stress damage. (Section 2.3).

A similar approach was used by Contreras to determine the probability of slope failure. (Contreras 2015).
2.3 Damage loss model

The estimation of losses associated with stress damage and rockfalls in underground mines has been addressed by a few authors (Joughin et al. 2012a; Joughin et al. 2012b; Abdellah et al. 2014). The economic consequences of the damage must be considered when estimating the potential losses. Typical consequences are:

- Remediation of the damaged section of the tunnel
- Lost production due to inaccessibility
- Damage to machinery
- Lost production due to lack of availability of machines, if they are trapped or damaged.

The economic losses associated with each of these consequences, depends largely on the function of the tunnel and the length of tunnel affected by the damage. Figure 17 shows the example mining layout with different types of tunnels servicing the stope production: Access ramp, Sub-level drive, Primary stope drive and Secondary stope drive.

Remediation usually involves the removal of loose and damaged rock and re-supporting. It is probably best to obtain estimates of the remediation cost per metre of tunnel (r) from previous tunnel repairs. The value of r will also depend on the function and service life of the tunnel, as well as local safety regulations and different values of r should be determined for different types of tunnels.

When considering the production losses, the average daily production (tonnes/day), which is served by the tunnel under consideration, must be estimated. The potential daily revenue loss should then assessed using the value per tonne of ore. This will also differ depending on the function of the tunnel. It can be assumed that no production can be achieved, until the tunnel is rehabilitated. Referring to Figure 17, damage in the main access ramp would always affect the full production from these stopes, while damage
in the sub-level drive would probably only effect half of the production. If the primary stopes are being mined during the period under consideration, then damage in the secondary stopes would probably not affect production at all, providing that they are remediated before the secondary stopes are mined.

The mine planning engineer can estimate the typical impact on daily production for each type of tunnel during the period under consideration. The duration of the production loss can be determined by reviewing typical time it takes to remediate stress damage and rockfalls in tunnels and estimating the average number of days per linear metre of tunnel to be re-supported. If there is some flexibility in the system, the production loss may only occur if the tunnel is inaccessible for a long period. There may also be a lag time before the support crew can be mobilised, which needs to be taken into consideration.

Machinery may be damaged, if it happens to be travelling through the area where the fall occurs, at the time it occurs. The probability of spatial coincidence can be simply estimated as the length of the machine divided by the length of the tunnel. Time exposure of machinery in the tunnel must also be taken into consideration. The cost of damage can be estimated by reviewing the historical damage and maintenance costs and determining an average cost per incident. Machinery may also be trapped beyond the rock fall. The lack of availability could have an impact on production.

Simple models can be developed for different types of tunnels taking into account the size of the rockfall (Section 2.1) or the length of tunnel affected by stress damage (Section 2.2). Strategies can also be implemented to limit the impact on daily production as a result of damage, based on this thinking.

**Figure 17** Plan view of example mining layout indicating the impact of damage on production

### 2.4 Risk Evaluation

The economic risk can be assessed by determining the probability of occurrence of damaging incidents and then evaluating the economic losses associated with these damaging incidents. The Expected loss (E) represents the most likely damage cost as a result of all potential damaging incidents:
Where $p_i$ and $l_i$ are the estimated probability of occurrence and loss for each potential incident, taking rockfalls (Section 2.1), stress damage (Section 2.2) and the damage loss model (Section 2.3) into consideration.

Table 1 provides an indication of the expected losses for the access ramp and primary stopes drives. It is apparent that the access ramp poses a greater risk, since any significant damage will affect all production.

<table>
<thead>
<tr>
<th>Tunnel</th>
<th>Rockfalls</th>
<th>Stress Damage</th>
<th>Total</th>
</tr>
</thead>
<tbody>
<tr>
<td>Access ramp</td>
<td>$1.92M</td>
<td>$2.41M</td>
<td>$4.33M</td>
</tr>
<tr>
<td>Primary stope drive</td>
<td>$0.02M</td>
<td>$1.78M</td>
<td>$1.80M</td>
</tr>
</tbody>
</table>

When considering alternative support systems, the total cost of the support systems should be compared. The total cost of a support system ($T$) can be determined as follows:

$$T = S + I + E$$

Where $S$ is the material cost of the support and $I$ is the installation cost.

This provides a single value for a support system analysis, which can be compared with other support system analyses. This approach is valid for events which occur relatively frequently and have a low impact.

Events that are very unlikely, but have a major impact need to be treated differently. An alternative approach is to use a risk matrix or a risk map (Abdellah et al. 2014; Contreras 2015). Most mining operations use a risk matrix system to apply judgement in the assessment of risk and decision making. Figure 18 represents a typical risk matrix. The probability of occurrence and the value of the potential losses form the axes of the matrix. The level of risk can then be mapped. These risk matrices vary from operation to operation, depending on the level of financial risk tolerance.

<table>
<thead>
<tr>
<th>Probability of Occurrence</th>
<th>Damage Loss</th>
</tr>
</thead>
<tbody>
<tr>
<td>Insignificant &lt;$0.01M</td>
<td>Low</td>
</tr>
<tr>
<td>Minor $0.01M-0.10M</td>
<td>Low</td>
</tr>
<tr>
<td>Moderate $0.10M-1.0M</td>
<td>Low</td>
</tr>
<tr>
<td>Major $1M-$10M</td>
<td>Low</td>
</tr>
<tr>
<td>Catastrophic &gt;$10M</td>
<td>Low</td>
</tr>
</tbody>
</table>

Figure 18  Risk Matrix chart example

Table 2 presents suggested probabilities to be assigned to the subjective, qualitative description of likelihood.
A risk-based approach to ground support design

W.C. Joughin, J.J.M. Muaka, P. Mpunzi, D. Sewnun and J. Wesseloo

Table 2  Suggested probabilities

<table>
<thead>
<tr>
<th>Probability Description</th>
<th>Criteria</th>
<th>Probability</th>
</tr>
</thead>
<tbody>
<tr>
<td>Certain</td>
<td>The event will occur. The event occurs daily</td>
<td>&gt;50%</td>
</tr>
<tr>
<td>Likely</td>
<td>The event is likely to occur. The event occurs monthly</td>
<td>10% to 50%</td>
</tr>
<tr>
<td>Possible</td>
<td>The event will occur under some circumstances. The event occurs annually</td>
<td>5% to 10%</td>
</tr>
<tr>
<td>Unlikely</td>
<td>The event has happened elsewhere. The event occurs every 10 years</td>
<td>1% to 5%</td>
</tr>
<tr>
<td>Rare</td>
<td>The event may occur in exceptional circumstances. The event has rarely occurred in the industry.</td>
<td>&lt; 1%</td>
</tr>
</tbody>
</table>

The results of the analysis can be presented on a risk matrix as shown in Figure 19. The potential risk can be communicated in a manner that can be interpreted by management according to the operational risk policy. In the examples, it is apparent that the risk does not reach extreme levels, but is high. The access ramp again shows a much higher level of risk, since it affects all production when significant damage occurs. Primary stope drives are more likely to experience damage, but the impact on production is significantly lower. The risk could be mitigated by improving support or modifying the mining layout and sequence.

![Risk Matrix Diagram](image-url)

Figure 19: Results of analyses presented on a risk matrix
3 Additional factors to consider

In reality, there are many additional factors that can affect the potential risk, some of these are:

- Incomplete geotechnical data
- Scale variability
- Uncertain stress field
- Influence of major geological structures
- Time dependent deterioration
- Model bias (simplifications and assumptions)
- Human error during implementation

Invariably, there is a paucity of geotechnical data, particularly when comparing with resource models. Measures of confidence will always be low when compared with the requirements for resource estimation. In many cases this is acceptable, because the risks can be addressed in other ways, but in some circumstances it may be necessary to collect more data and report levels of confidence. In this example, scale variability is appropriately taken into consideration for GSI, but not for UCS. This is largely due to the standard test methods and the cost of testing. Geophysical methods could be possibly used to estimate the variability in UCS and scale appropriately.

Some of the other factors can be modelled by including them as additional variables, but it is then necessary to apply engineering judgement and rely on experience to estimate the ranges of these variables. Triangular or uniform distributions should be used due to the lack of information.

4 Conclusion

This paper describes a preliminary risk-based approach to ground support design which incorporates the mechanisms of block failure and stress damage, utilises probabilistic solution techniques, incorporates a damage loss model and allows the risk to be quantified and evaluated.

The influence of any number of input variables can be taken into consideration. However, it is important to consider Occam’s razor and not make the analysis overly complex. There will always be some epistemic uncertainty, which cannot be rigorously assessed. The philosophy provided by Vick (Vick 2002) where he distinguishes between a “relative frequency approach” and a “subjective, degree of belief approach” is useful:

- Relative frequency approach: The probability of an uncertain event is its relative frequency of occurrence in repeated trials or experimental sampling of the outcome.
- Subjective, degree of belief approach: The probability of an uncertain event is the quantified measure of one’s belief or confidence in the outcome, according to their state of knowledge at the time it is assessed.

The relative frequency approach is used by the insurance industry, where large amounts of data are available and it is possible to assess the potential risk quite reliably. In geotechnical engineering, there is often incomplete data and epistemic uncertainty. The subjective degree of belief approach is therefore more appropriate. Where good data is available, this will improve confidence in the outcome and the degree of belief, but the interpretation will remain subjective.

Most importantly the results need to be effectively communicated. Risk matrices are commonly used on operations and it is therefore useful to present the results in this format.

It should be noted that the safety risk is also important to consider. A method of estimating the frequency of injuries and evaluating safety risk is proposed in (Joughin et al. 2012a; Joughin et al. 2012b).
Acknowledgement

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Advanced 3DEC bolt model for simulation of ground support performance in highly fractured and bulked rock masses

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Abstract

The design of effective ground support is critical to the success of the next generation of large cave mines. Typically the behavior of rock in this setting is controlled by shear and opening movements along fractures induced by the cycles of loading and unloading that accompany mine development and operation. Itasca Consulting Group has recently conducted a review of numerical modeling procedures for support of highly stressed jointed and/or fractured rock. This review and a recent literature study of laboratory scale experiments has revealed two important observations in ground support design: (i) in blocky or fractured rock the majority of the support deformation is localized at joints or fractures and (ii) the resistance to fracture shear displacement offered by support is important. These observations have led to use of the discrete element (DEM) based 3D Bonded Block Model (BBM) to represent the rock and a new hybrid bolt model to represent the bolt support. The hybrid bolt model is an improvement on the classical cable bolt model by offering more realistic resistance to fracture shear displacement. This paper (i) gives a review of relevant laboratory experiments for steel rebar, (ii) describes the hybrid bolt model, and (iii) describes model calibration to this data.

1 Introduction

Massive underground mining is becoming ever more prevalent, specifically caving mines at depths greater than 1000 m. Tunnel support designs must consider the development process of cave mine infrastructure in deep, highly stressed environments. Mine development sequencing and cave advance affect the redistribution of induced stresses on mine infrastructure. Support designs must consider the excavation instability problems resulting from associated loading and unloading. Garza-Cruz and Pierce (2014) have shown that the discrete element method (DEM) is preferable to continuum approaches for representing the spalling and bulking which occur at depth. Discontinuum approaches, such as DEM, can explicitly represent the initiation and propagation of fractures. DEM models show brittle failure is dominant at low confinement while brittle failure and the propagation of tensile fractures is inhibited at higher confinement. This mechanism is important for accurate stability assessment and support design.

3DEC (Itasca Consulting Group, Inc. 2013) allows for the construction of synthetic rock samples with zero initial porosity. The model consists of a collection of interlocked tetrahedral blocks bonded at their contacts. The tensile strength of these contact bonds is heterogeneous. A bond breaks when the bond tensile or shear strength is exceeded. After a contact is broken, it resists joint shear motion via Coulomb friction. The block contacts represent a network of low persistence veins, open fractures and intact rock while the blocks themselves represent unveined intact rock. Such models are referred to as Bonded Block Models (BBM). A rock bolt model was recently added to 3DEC to enable BBM models to study support effectiveness in this setting. This was challenging for two reasons. First, the mechanical behavior of bolted rock fractures is very complex and not fully described in the literature despite significant research. A
numerical implementation has to be simple enough to be computationally manageable in a DEM simulation at the tunnel scale while able to reproduce the key components of the reinforcing effect of the bolt considering a reasonable number of input parameters. Secondly, in a BBM model, such support has to be installed in an already open jointed/fractured rock mass due to the relaxation of the tunnel at the time of the support installation. A new structural element developed in 3DEC called the “hybrid bolt” is able to reproduce both the shear and axial behavior of fully grouted rebar, cable bolts and split sets. The model is a combination of a traditional cable element to model the axial behavior and local shear spring at fracture intersections to model the shear resistance induced by the presence of the bolt system (steel and grout). Both elements can yield and rupture and can reproduce the 3-stages of load response (elastic, yield and plastic stages) typically observed in laboratory tests.

Section 2 presents a literature review of bolt behavior under simple laboratory tests. Section 3 details implementation of the new bolt model in 3DEC. Finally, Section 4 presents the model calibration to simulate a fully grouted rebar.

2 Literature review

The purpose of the literature review and the selected extracts is to have a better qualitative understanding of the fracture1 reinforcement provided by a bolt in shear and the failure mechanism of the bolt under shear; as well as to assemble laboratory test results that can be used to calibrate the 3DEC structural element so that performance in both pull-out and shear can be correctly handled.

2.1 Reinforcement Mechanism of a Bolt

Bolted fractures can be subjected to two categories of movements, either to opening of the fracture in a direction perpendicular to the plane or to shear displacements occurring in the plane. Generally, deformation in fractured or stratified rock combines both types of movements because of the dilation of rough fractures.

According to Dight (1982), the resistance provided by a bolt crossing a fracture in both shear and opening (due to dilatancy) can be attributed to five types of forces:

- An increase in shear resistance, R1, due to lateral resistance developed by the rock bolt via dowel action.
- An increase in normal stress, R2, as a result of pre-stressing of the rock bolt.
- An increase in normal stress, R3, as a result of axial force developed in the rock bolt from dilatancy of the fracture.
- An increase in normal stress, R4, as a result of axial force developed in the rock bolt from lateral extension.
- An increase in shear resistance, R5, due to axial force in the rock bolt resolved in the direction of the fracture.

These five types of forces can be divided in two components: cohesion and friction. R1 and R5 increase the cohesion component of the fracture shear strength, while R2, R3 and R4 increase the friction component by increasing the effective normal stress. Without friction on the fracture, those terms are null. Spang and Egger (1990) explain that the normal force component inducing the friction effect is the result of the bending of the bolt in deformable media and/or its inclination. They suggest that the friction effect may

1 Note that we prefer to use the term ‘fracture’ instead of ‘joint’. Indeed, the term ‘joint’ is specific to pre-existing planar fracture in the rock mass. The generic term ‘fracture’ applies to pre-existing joints and also emergent fractures newly created. In BBM models, both types can be encountered since those models are able to simulate initiation and propagation of fractures.
increase the shear resistance of the bolted fracture up to 130% of the maximum tensile load of the bolt. They call ‘dowel effect’ what Dight defines as the cohesion component, the component of the force parallel to shearing and suggest it is equivalent to 70 to 80% of the maximum tensile capacity of the bolt.

2.2 Experimental results of fully grouted rebar shear and pull-out tests

This section presents and discusses several laboratory test results for a common type of rock bolt: 20 mm diameter fully grouted rebar. Those results are used as a reference to describe the generic behavior of rock bolts in shear and also to calibrate the 3DEC hybrid bolt element in Section 4.

2.2.1 Comparison of experimental results

Table 1 summarizes the configuration of each of the tests presented in Figure 1.

Grasselli (2005) showed that the effect of two bolts on load is exactly double the effect of a single bolt so to reflect results of a simple shear test on a single bolt, the load recorded by Aziz and Jalalifar (2007) is divided by 2 and the load recorded by Grasselli (2005) for the double shear test on two rebars is divided by 4. Results for “inclined” bolts from Chen & Li (2014) are presented here as total load versus shear displacement (calculated from the total displacement based on the formula available in the reference) and not the total displacement as presented by the authors in the reference. Note that to reproduce the bolt inclination effect, Chen & Li (2014) apply a combination of shearing and opening to a perpendicular bolt and define a ”displacing angle” – this is different from simple or double shear test on an inclined bolt as performed by Grasselli. Those results have to be interpreted with caution.
When comparing the various experimental shear tests shown in Figure 1, the following observations can be made. Although test set-ups may vary (in terms of grout properties, hole diameter, fracture friction etc.), there is a consistency between Aziz and Jalalifar (2007), Chen and Li (2014) and Stjern (1995) shear tests (although a significantly smaller maximum displacement is recorded by Aziz and Jalalifar). Thus, the main parameter controlling the bolt behavior under shearing seems to be the rebar diameter (20 mm for those tests). The large scale test performed by Deveaud (2012) shows significant higher stiffness both for elastic and plastic stages, but similar strength. It also shows a smaller maximum displacement. The Grasselli shear tests on 16 mm rebar show a similar behavior to other tests performed on 20 mm rebar with lower stiffness and strength as one would expect. The shear test performed on a 20 mm rebar, however, behaves quite differently from other tests although the experimental set up is similar. This test actually shows similar behavior to pull-out tests. We have not been able to explain this behavior; as a result, tests from Grasselli are not taken into consideration for the rebar behavior calibration.

When comparing the various experimental pull-out tests shown in Figure 1, the following observations can be made. Tests performed by Stillborg (1994) with cement and resin grouted rebars show very similar results. The type of grout doesn’t seem to be of primary influence. The Set 2 tests performed by Chen & Li (2014) is close to Stillborg results but show higher stiffness during the initial elastic behavior as well as a higher shear resistance. The behavior during the plastic stage is very similar. The Set 1 tests performed by Chen & Li (2014) are very similar to Set 2 except the initial displacement of 4mm recorded without any resistance. This might be due to a recording problem or an incorrect bolt installation.

Figure 1  Simple shear and pull-out experimental tests results performed on a grouted untensioned rebar perpendicular to the fracture
2.2.2 Description of stages during shear test

As seen in Figure 1, in most cases three stages of the rock bolt system behavior can be distinguished when subjected to direct shearing and have been well described by Spang & Egger (1990) and Grasselli (2005). These three stages are the elastic stage, the yield stage and finally the plastic stage; they are all described below in detail. Depending on the type of reinforcement, each stage has a different “weight” on the final behavior.

First, the elastic stage corresponds to a linear behavior associated to small displacements and great increase of load. The cohesion of the fracture is overcome and the blocks begin to slide. Stresses are compressive on the side behind the bolt and tensile in front side as shown in Figure 2. The tension stresses in the mortar and the rock will soon disappear because of the very low adhesive strength between steel and mortar giving rise to a gap on the tension side near to the fracture plane. For example, for the test performed by Grasselli (2005) on a 20 mm rebar, the elastic stage corresponds to the beginning of the test from 0 to 1.5 mm of joint shear displacement.

Then, a non-linear behavior is observed caused by yielding of the materials; this is called the yield stage. In the case of extremely rigid surroundings, the bolt resists until its shear strength is reached and then fails. In the case of deformable surroundings (for weaker rocks and typical bolts in a mortar annulus half a diameter thick), the bolt has to be deformed in order to mobilize shear resistance. As a consequence of these deformations, the yield strength of the steel and of the mortar are reached by bending and by compression, respectively. This occurs at very low shear displacements (a few millimeters) and forces (around 10% of the bolt ultimate value). The yield stage is governed by the yield limit of the steel and by the compressive strengths of the mortar and the rock. For example, for the test performed by Grasselli (2005) on a 20 mm rebar, the yield stage corresponds to the part of the test for a joint shear displacement from 1.5 to 10 mm.

Finally the plastic stage is observed when all of the materials yield. The shear response of a bolted fracture essentially depends on the force-displacement relationship of the plasticized materials. The plastic stage corresponds to an accumulation of joint shear displacement without significant increase of the load. Finally, the nearly unconstrained plastic deformation of the bolt leads to its failure. A.M. Ferrero (1995) suggests that two different failure mechanisms can be observed. For hard rock (UCS > 50 MPa), failure is usually due to a combination of shear and tensile axial stresses acting at the bar-fracture intersection. For weaker rock (UCS < 50 MPa), failure is due to a combination of tensile stresses and bending moments. Indeed, as the rock strength becomes weaker, the failure mechanism changes with the formation of two plastic hinges symmetric with respect to the shear plane. The steel is subjected to larger strain and failure occurs due to the axial force (reaching ultimate tensile load or ultimate steel strain) between the two plastic hinges. For example, for the test performed by Grasselli (2005) on a 20 mm rebar, the plastic stage corresponds to the part of the test for a joint shear displacement above 10 mm.

In some cases one of the stages described above can be missing: for example if the surrounding rock is too weak and starts yielding at the beginning of the test; or if the bolt fails right at the end of the yielding stage without showing any significant plastic stage.

Figure 2 Bolt system (a) in the field (from Li, 2009), (b) as a diagram and (c) modelled in 3DEC
2.3 Support effectiveness

This literature review identified a number of important points in the context of studying support effectiveness of tunneling at depth in a heavily fractured and bulked rock mass. In the specific case of a bolt perpendicular to a fracture in a stiff rock mass, the friction effect can be neglected and only the dowel effect acts on the fracture. Otherwise, in a vast majority of cases, both effects act on the system and their relative importance on the bolt capacity to reduce fracture displacement depends mainly on the bolt orientation, the fracture friction, and the stiffness of the various materials. The fracture friction significantly influences the maximum resistance carried by the bolt. The difference between a low and high friction fracture can be as much as 50%. Without pretension or dilatancy, the bolt usually has to be sheared significantly (dowel effect) before starting to offer significant resistance in tension (friction effect). The more the bolt is perpendicular to the fracture, the more this is true. Both pre-tension of the cable and dilatancy of the fracture have similar effects on the bolt: they “activate” the friction effect at an earlier stage of the fracture mobilization (i.e., at smaller displacements) when compared to a non-prestressed bolt or non-dilatant rock. It can be positive in terms of support in the first place, by reducing the displacement of the fracture for a given load. But it also can lead to an early failure of the bolt (and so a complete loss of its efficiency), because the tensile load capacity of the bolt can be reached at smaller displacement and load. The stiffness of the system is as important as the bolt strength. Indeed, the maximum load capacity of the bolt is crucial, but the fracture displacement associated to it is also important. This displacement depends strongly on the bolt orientation and the rock mass stiffness. The stiffness of the system, which controls how the stress increases with displacement at the different stages of the loading, also influences the failure mode of the bolt. Is the tensile load capacity reached first or the maximum strain?

3 Bolt model in 3DEC

Bolts crossing fractures will provide some resistance to shearing on the fractures. Of the five different mechanisms suggested by Dight (1982) (see Section 2.1), R1 increases the shear resistance due to lateral resistance developed by the rock bolt via dowel action while R2 to R5 result from the axial force developed in the rock bolt. In 3DEC, cable elements alone will demonstrate effects R2 to R5, but not R1. The dowel action is really an umbrella term for a set of complex mechanical effects including bending of the steel bolt, crushing of the grout, crushing of the host rock, etc. We make no attempt to reproduce all of these effects in 3DEC. Instead the dowel effect, not simulated by the cable elements, is simply represented as an additional shear spring that resists slip on the fracture where it is crossed by the bolt as shown in Figure 2. The shear spring has an elastic-perfectly plastic behavior which allows for it to yield and fail. Although these are fairly simple elements compared to the mechanism they are meant to reproduce, it will be shown in the following sections that with proper calibration the 3DEC bolt is able to properly reproduce the rock bolt response in both opening and shear. Node placement is an important implementation detail, this is discussed in Sections 3.1 and 3.2. Details about the classic cable element implementation can be found in the 3DEC Theory & Background manual (Itasca Consulting Group, Inc. 2013).

3.1 Automatic node placement

With cable elements, the number of segments along the length of the cable is given with the seg keyword. With hybrid bolts, it is recognized that the distribution of nodes around fractures has an effect on the bolt behavior. Therefore more control of the node spacing is provided using two keywords slen and dlen. The keyword slen dictates the length of cable bolt segments inside blocks (between fractures) and dlen gives the length of bolt elements that cross fractures. If dlen is not given, then it defaults to slen. An example is shown in Figure 3a. If a block is smaller than 0.1×dlen, no node is placed in the block. If a block is larger than 0.1×dlen but smaller than 2×dlen, then a single node is placed in the middle of the block. The dlen parameter is also the length of the dowel segment. The length of the dowel segment is only used in the calculation of shear strain (=shear displacement / dlen). It has no effect on the calculated forces. Figure 3b presents an example of hybrid bolt geometry in the wall of a tunnel in a BBM.
3.2 Shear behavior at fracture intersection

The dowel shear force-displacement relation is described in incremental form by the expression:

\[
\Delta F_s = K_s |u_s| \tag{1}
\]

where:

- \(\Delta F_s\) = an incremental change in shear force;
- \(|u_s|\) = an incremental change in shear displacement;
- \(K_s\) = the input shear stiffness.

The shear forces developed in the dowel are applied to the faces of the blocks on either side of the fracture by interpolating to the zone face nodes. It is important to note that the dowel formulation only works with deformable (zoned) blocks. A yield strength can be specified for the shear force. When the yield strength is exceeded, the dowel behaves plastically and the shear resistance remains constant as shear displacement continues. A rupture limit (strain) may also be specified. If the relative shear strain in the reinforcement element exceeds the specified strain limit, then the shear force will be set to zero. In this case, the host cable will also rupture at the same location and its axial force will be set to zero. When the host cable ruptures (the input axial strain limit is surpassed), the dowel also ruptures at the same location. The following assumptions should also be noted. There is no axial component to the dowel spring. This is because it is assumed that the cable itself will provide axial forces. The shear spring is always oriented parallel to the fracture, regardless of the cable orientation relative to the fracture. This is to avoid spurious axial components of force.

4 Modelling of a fully grouted rebar

Section 4.1 presents the calibration of the bolt model in 3DEC to represent fully grouted rebar of 20 mm diameter based on simple shear and pull-out experimental tests. Section 4.2 shows the behavior of the model with changes in inclination and friction; a comparison to experimental observation is given.
4.1 Simple shear and pull-out tests

The 3DEC hybrid bolt parameters have been calibrated to be able to reproduce the behaviour of rebar in shear and pull-out tests. The experimental results used as a reference for this calibration are those presented in Section 2.2. Figure 4 presents the corresponding simple shear and pull-out test results. The calibrated properties for rebar are: Cable Young’s modulus ($emod$) 1.4e11, node spacing ($slen$) 10 cm, grout stiffness ($kbond$) 3e8 N/m/m, grout strength ($sbond$) 2.8e5 N/m, axial yield strength ($yield$) 1.8e5 N, axial rupture strain ($strain_limit$) 0.2, dowel shear stiffness ($dowel_stiffness$) 1e7 N/m, dowel shear strength ($dowel_yield$) 6.3e4 Pa, shear rupture strain ($dowel_strain_limit$) 0.4.

The solid blue curve in Figure 4 shows the pull-out force-displacement curve predicted by the 3DEC model calibrated for a rebar bolt. The dashed blue lines show a summary of fully grouted rebar laboratory pull-out tests. Three stages can be identified similar to what is commonly observed during experimental tests. First, the elastic stage shows a rapid increase in load. Displacement results in an increase of force in the cable element. The emergent modulus is a combination of the cable stiffness ($emod$) and the grout stiffness ($kbond$). This stage ends at about 3 mm of displacement when the grout interface between the rock and the cable locally reaches its yield value ($sbond$) simulating the debonding of the grout/rock interface or the reinforcing/grout interface. During the yield stage, as the pulling continues, the force in the cable element keeps increasing and the grout yielding propagates along the cable. The slope depends on the cable element stiffness ($emod$), the grout stiffness ($kbond$) and the grout yield value ($sbond$). The weaker the grout, the faster the grout yielding propagation along the cable and the softer the response. This stage ends when the cable element reaches its yield value ($yield$) of 180 kN. During the plastic stage the cable element has reached its yield force so the load remains constant while it stretches. The central cable element continues to yield until the specified rupture strain ($strain_limit$) is reached. When it does at about 30 mm of displacement, both the cable element and the dowel element forces are automatically set to zero to model the rupture of the bolt. Note that hybrid bolt behaviour in pull-out test only depends on properties associated to the cable behaviour; the “dowel” part of the logic only acts when the bolt is sheared.

The solid green line in Figure 4 shows the shear force-displacement curve predicted by the 3DEC model calibrated for a rebar bolt. The dashed green lines show a summary of fully grouted rebar laboratory shear tests. Three stages can be identified for the shear test similar to what is observed during experimental tests. First, the elastic stage shows a rapid increase in load. Shear displacement results in an increase of force in the dowel and the cable elements. The emergent modulus is a combination of the cable stiffness ($emod$), the grout stiffness ($kbond$) and the dowel stiffness ($dowel_stiffness$). This stage ends when the dowel element reaches its yield force value ($dowel_yield$) at about 7 mm of shear displacement. Yielding of the dowel reproduces the slope change observed on experimental tests corresponding to the crushing of the grout and the rock at the intersection between the bolt and the shearing fracture. During the yield stage, as the shearing continues, only the force in the cable element keeps increasing. The load increase depends on the element orientation: the more it is inclined, the faster the load increases (geometric effect). This is why the curve’s slope increases with the shear displacement. The slope depends on the cable element stiffness ($emod$) and the grout stiffness ($kbond$). The slope decrease at 22 mm of displacement occurs when the cable grout interface starts yielding close to the fracture so that any increase in shear displacement is accommodated by a longer length of the bolt. This stage ends when the cable element reaches its yield value ($yield$) which corresponds to a bolt resistance of 220 MPa. During the plastic stage, both the cable element and the dowel element have reached their yield force so the load remains almost constant until the dowel element reaches the specified rupture strain ($dowel_strain_limit$) at 41 mm of shear displacement. When it does, both the cable element and the dowel element forces are automatically set to zero to model the rupture of the bolt.
4.2 Bolt Inclination and Fracture Friction Effect

Most laboratory experiments perform simple shear tests and pull-out tests on a bolt perpendicular to the fracture. But in the field, rock bolts can be subjected to a combination and/or succession of shear and axial mobilization in various directions. Only a few references study the effect of bolt inclination on the bolt behavior when crossing a fracture. The hybrid bolt model in 3DEC is not meant to reproduce all the complexity of rock bolt behavior in every possible condition. It is meant to consider the shear resistance generated by the bolt at fracture intersections, simply enough to be implemented and simulated in a reasonable amount of time for a large scale tunnel model while reproducing accurately the support brought to the rock mass estimated experimentally. The hybrid bolt model properties are calibrated through simple shear and pull-out tests on bolts perpendicular to the fracture. All the parameters are then fixed. This section discusses the inclination effect on a simple shear test in 3DEC for a given set of parameters, and compares it to observations available in the literature. There is no attempt to take into account the inclination effect during the calibration process as because there is very little experimental or field observation values available to compare to and because it would make the process too complex for the purpose of its application. The reviewed literature agrees on the following points about bolt inclination and friction effect:

- The maximum shear strength of the bolted fracture increases with the fracture friction angle.
- For a given fracture friction angle, the maximum shear strength of the bolted fracture varies with the bolt inclination.
- Inclined bolts react in a stiffer way than normal ones.
- The greatest ultimate displacement varies and is reached when the bolt is normal to the fracture.
- Confining effect increases with inclination while the cohesion term decreases.

During a simple shear test, the 3 forces contributing to the bolt effect on the fracture are the dowel element shear resistance $F_d$, the cable shear resistance $F_{CS}$ and the additional friction force due to the cable normal force $F_{CN}$. If we consider a bolt inclination of $\theta_1$ (so that $90^\circ$ corresponds to a bolt perpendicular to the fracture) and a fracture friction $\phi$, the maximum bolt contribution equals:

$$\text{Maximum Bolt contribution} \cong F_d + F_c \cos \theta_1 + F_c \sin \theta_1 \times \tan \phi$$

(2)
where:

$$F_c = \text{Axial force acting in the cable element whose maximum value is the input cable yield value}$$

$$F_d = \text{Force acting in the dowel element whose maximum value is the input dowel yield value}$$

As a consequence, the hybrid bolt contribution in 3DEC depends on the friction angle and the bolt inclination. Considering that the bolt rotates a few degrees during the simple shear test, Figure 5a gives an estimation of the 3 forces generated by the presence of the hybrid bolt for various bolt inclination and for different friction angles.

![Figure 5](image)

Figure 5 (a) Variation of various bolt actions contribution in 3DEC with bolt inclination for five different fracture friction angle (0°, 20°, 30°, 40°, and 50°) and (b) Bolt inclination effect on simple shear test with a fracture friction angle of 40° (Inclination =90° for a bolt perpendicular; inclination >90° for bolts inclined in the opposite direction to shearing).

Figure 5b shows simple shear test results in 3DEC for a friction angle of 40° and various bolt inclinations (from 10° to 130°). As observed experimentally for rock bolts:

- The maximum shear strength of the bolted fracture varies with the fracture friction angle; the higher the friction, the higher the maximum shear strength. For a given fracture friction angle, the maximum shear strength of the bolted fracture varies with the bolt inclination. For a 40° friction angle, the maximum value is observed around 45° and corresponds to a 30% increase compared to perpendicular bolt.

- Inclined bolts react in a stiffer way than normal ones. This is due to a geometric effect. The more the bolt is inclined (compared to the perpendicular direction to the fracture), the greater the increment of axial force for a given increment of shear displacement.

- The friction effect $F_{fb}$ in Figure 5a increases with inclination while the cohesion term (i.e., dowel action $F_d$ and shear cable action $F_{cs}$ in Figure 5a) decreases.

- The ultimate shear displacement leading to bolt failure varies with the bolt inclination. It is maximum when the bolt is perpendicular to the fracture. In this case, it is directly related to the input dowel shear rupture strain (maximum shear displacement = maximum dowel strain x dowel length). When the bolt is more inclined ($\theta>70°$ in Figure 5b), the cable strain limit for axial rupture is reached before the dowel strain limit since the cable element gets elongated more quickly. And the more the bolt is inclined, the less fracture shear displacement is necessary for the cable to reach the axial strain limit.
When the bolt is inclined in the opposite direction to shearing, the cable element gets compressed while the fracture is sheared (and not stretched) so that the only force applied by the bolt on the fractures comes from the dowel element. At the end of the elastic stage, when the dowel reaches its yield value, the force no longer increase except if the cable element rotates enough so that it gets stretched again (this is the case at the end of the test for Θ=110°).

Very few experimental results are available considering inclined bolt subjected to shearing. In terms of qualitative behavior, the 3DEC hybrid bolt reproduces most effects due to inclination described in the literature for analytical or experimental results. In terms of quantitative results, we believe the hybrid bolt behavior is reasonable considering the level of accuracy it is meant to achieve and in the order of magnitude observed in the few available experimental tests.

5 Conclusion
The cable model implementation in 3DEC has been updated to model the shear resistance of bolts at fracture intersection in addition to the axial resistance brought by the cable model. The literature review showed that rebar bolts with identical diameter displayed similar resistance to fracture shear displacement despite several set-up discrepancies such as rock mass properties or grouting. Thus it is reasonable to use a single rebar model solely based on the diameter in studying the reinforcement effect. Simple-shear and pull-out numerical tests were conducted to calibrate the bolt model to reproduce the 20 mm fully grouted rebar behavior observed experimentally. It has been shown that the 3DEC bolt model is able to properly reproduce the 3 main stages occurring when a bolt is sheared through a fracture as well as the all the reinforcement mechanisms described in the literature related to the friction effect and the dowel effect. Inclination and friction influence on the bolt model behavior has been studied and found in accordance with experimental observations. The application of the 3DEC bolt model in a BBM tunnel model is currently being studied to assess tunnel performance at depth. The results will be presented in a subsequent paper.

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An experimental study to investigate the interaction of backfill and rock mass

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Abstract

The economic, ecological and safe extraction of minerals from underground often requires the application of backfill. Currently the design and operation of backfill systems is largely based on practical experience rather than on well-researched engineering principles. This can lead to misjudgements of backfill performance with serious consequences concerning mine safety, mining system performance, mine profitability, and mineral deposit exploitation.

To obtain a better understanding of the interaction of backfill with the surrounding rock strata, a research project in an Austrian underground mine using backfill was launched. The investigated mine is an underground magnesite mine using a system of large stopes separated by slender pillars and filled with cemented backfill as mining method in the considered mining areas. In the up to 1000 m deep sections a system of 21 m high, 6 m wide and up to 80 m long stopes separated by 7 m wide pillars is applied. Only one stope is mined in a section at any time. After the stope has been backfilled the adjacent stope is excavated. This mine offers the opportunity to evaluate the influence of backfill on rock mass behaviour during different operation steps.

The focus of research was on investigating the effects of backfill on rock deformation and fracturing processes around mining excavations. Characteristic reference areas were selected based on geological and stress conditions and planned mining operations. The geology of the areas was carefully mapped, in situ discontinuity patterns analysed based on conventional discontinuity measurements and core drilling. Rock mass properties (laboratory and in situ) and backfill properties (laboratory and in situ) were determined. In addition, stress build up in the backfill was monitored in three directions using flat jack type pressure cells. Rock deformations and fracture patterns around mining excavations were monitored before, during and after placement of backfill using multiple rod extensometers and optical borehole observations. Similarly seismic velocity measurements to quantify changes in rock conditions were carried out. Particular attention was given to assess and document the effects of stoping activities on rock deformation around the mining excavations. This was necessary to account for the mining induced stress changes in the areas of interest.

The paper describes the results of the comprehensive programme of rock mechanics investigations. The results show that backfill placement in stopes has an immediate effect on lateral deformation of slender stope pillars. This is despite the fact that stress build up in the backfill is rather slow. This result is of practical significance as it puts into question some of the backfill requirements specified by the Austrian mining authority. Monitoring of rock deformation around mine tunnels several tens of metres away from the stoping area provides a good basis for assessing regional effects and assists in long-term stope planning investigations. An important outcome of the research project is that the instrumentation methodology used in this study has proven to be reliable, accurate and can be implemented by mining personnel.
1 Introduction

In deep mines when caving systems cannot be employed for environmental and/or safety reasons the maximum extraction possible from a deposit may be unacceptably low and safety hazards can arise as a result of mining activities. Backfilling of stopes can improve the situation as backfill can control stope wall deformations, minimise rock fall potential and increase local stability as well. Today most underground mines in Austria employ backfill. Currently the design and operation of backfill systems is largely based on practical experience rather than on well-researched engineering principles. As a result, misjudgements of the actual mining situation and backfill performance can develop with serious consequences concerning mine safety, mining system performance, mine profitability, and mineral deposit exploitation. For this reason, a research project investigating the influence of backfill on fracturing around underground excavations was launched. The scope of this project is to increase the knowledge of the interaction of backfill and rock mass, of the interaction of backfill and pillars and finally to determine the necessary physical properties of the backfill product to ensure the required behaviour of the rock mass. To achieve this goal, underground investigations (extensometer measurements, backfill stress measurements, core drilling, optical borehole surveying, and hammer stroke seismics) are carried out in an underground magnesite mine.

2 Mode of action of backfill

When the rock in the immediate vicinity of the excavation becomes overstressed, the fill material supports loosening material from the excavation boundary which is so kept in place thereby preserving the confining forces within the rock mass. Through backfill application, confinement on the roof and walls of the opening is increased, which prevents the opening of joints and fractures, as it mobilizes friction along the surfaces. This preserves rock mass shear strength. Backfill is particularly effective in controlling rock spalling as low normal stresses acting on the rock slabs formed by spalling prevent buckling of the slabs. Regarding the influence of backfill on pillar behaviour, the following effects have to be considered (Figure 1):

- It resists rock wedges sliding from pillar sides; hence backfill works against gradual disintegration of pillars.
- Passive backfill pressure increases the strength of very high and narrow pillars.
- Backfill offers a horizontal pressure on pillars, which works against lateral deformation of pillars and increases their resistance.
The passive backfill pressure equals the earth rest pressure for uncremented fill and its horizontal component is dependent on the height, density and angle of friction of the backfill body:

\[ \sigma_{pass} = \rho g h v K_{OH} \]  
\( \text{(Wagner 2009)} \)  

Where:

- \( \sigma_{pass} \): passive backfill pressure [MPa]
- \( \rho \): density of backfill body [kg/m³]
- \( h_v \): height of backfill [m]
- \( g \): gravitational acceleration [m/s²]
- \( K_{OH} \): coefficient of earth pressure at rest \( (K_{OH}=1-\sin \varphi) \)

This pressure prevents the sliding of rock wedges from pillars and confines the pillar thereby increasing its strength and changing the post-failure behavior of slender pillars from a brittle to a more plastic mode of failure. (Blaha and Wagner 2009)

Lateral pillar deformation and in particular pillar dilation is resisted by the backfill. This effect which is controlled by lateral deformation of the pillar, deformation properties of backfill and width of the stope is called the active backfill pressure:

\[ \sigma_{act} = \varepsilon_{lat} E_v \]  
\( \text{(Wagner 2009)} \)
An experimental study to investigate the interaction of backfill and rock mass

Where:

$$\varepsilon_{\text{lat}}$$...deformation of backfill under lateral load

$$E_v$$...deformation modulus of backfill [MPa]

$$\varepsilon_{\text{lat}} = \frac{\Delta \text{piller lateral}}{2 \text{piller width}}$$  \quad \text{(Wagner 2009)} \quad (3)

The lateral deformation of pillars compresses the backfill, which then develops a resistance against the deformation. The development of the active backfill pressure depends on the deformation of the pillar, the strength and deformation properties of the backfill body and the time of introduction of backfill. High initial stiffness and early introduction of backfill are the critical parameters. As far as the backfill system is concerned the overall backfill pressure acting on the pillar is the sum of the active and the passive backfill pressures. (Blaha 2012).

Different types of backfill are distinguished, according to the material used, the use of binding agents and the delivery method. The application of different backfill types depends on the properties required for the particular application reason. For higher strength of a backfill body, binding agents, mostly Portland cement are added to the mixture, increasing the cohesion between the particles. While there is general agreement on the above mentioned actions of backfill on support pillars there is very limited quantitative information on in-situ backfill effects in support pillars in deep mines.

3 Underground investigations

The underground mine, where the investigations take place, is located in the alpine region. Mineral deposits of the alpine type differ from those in many other mining districts. They tend to be geologically disturbed, resulting in irregular deposit shape, complex natural jointing systems and excessive faulting. As a consequence alpine mineral deposits tend to be relatively small which in turn results in small production panel sizes. Stress situations are usually complex and difficult to assess due to the tectonic history of the formation of the Alps and the irregular topography.

The mine selected for this study is typical of alpine mineral deposits and highlights the complex geological and mining situation encountered in the Alps.

3.1 Breitenau magnesite mine

The Breitenau magnesite deposit represents the largest known sparry magnesite deposit in the eastern alpine region. The deposit dips with 20-25° to the south and south-east and possesses a thickness between 50 and 200 m. In dip direction the deposit extends over a distance of two kilometres and in strike direction over a distance of up to 500 m. Characteristic features of this deposit are the bedding between two schist layers and the multi-phase tectonic history resulting in multiple discontinuities and faults. The mining activities are conducted in an open pit and an underground mine, with a total annual production of 400 000 t magnesite. The activities are distributed over five mining sections (Figure 2), with the mining method “Post pillar mining” in the upper four sections and at the lowest section a system of large rooms separated by slender support pillars whereby the rooms are backfilled with cemented backfill material. Depth of cover of the lowest section is around 1000 m.
The investigations of the current research work are concentrated in the lowest section of the mine. The mining geometry is as follows (Figure 3):

- Drifting of an up to 80 m long 6 m wide and 5 m high top drift.
- Drifting of an up to 80 m long, 6 m wide and 5 m high bottom drift. The two drifts are separated vertically by an 11 m temporary sill pillar.
- Extraction of the sill pillar.
- Filling of the stope with cemented backfill either up to the ground level of the top drift (when a second stope is planned above) or up to the roof of the top drift, which results in the complete filling of the stope.

Figure 2 Underground mine map Breitenau (Schenkl 2014)

Figure 3 Cross section of one stope at Breitenau mine: Drifting of bottom and top drift (left), sill pillar removal finished (middle), backfilled stope (right)

The stoping activities provide the unique possibility that all measurements and investigations can be conducted before, during and after sill pillar extraction. This means that all displacements and stress values can be directly linked to specific mining steps and so the influence of drifting, stoping and filling activities on local and regional displacements can be investigated.
3.2 Choice of reference areas

For the investigations underground, reference areas were chosen. Based on the following criteria the reference areas were selected:

- No or small influences from adjacent stoping.
- A homogeneous geological situation.
- Assured access to these areas during the project runtime.

In the Breitenau magnesite mine the following reference areas in mine section 6 were chosen (Figure 4):

- Stopes 600A-600G.
- Drift 600H.

![Figure 4 Mine map level 6 magnesite mine Breitenau](image)

The stopes 600A-600C were mined before the project start and stopes 600E-600G are mined during the project lifetime, whereas drift 600H is located outside the immediate mining area. According to the demands on the reference areas for the planned investigations, these sections were selected. The stopes 600A-600G represent homogeneous geological sections. These stopes are not under the influence of adjacent mining activities and access to the reference areas is assured over at least 3 years.

Additionally drift 600H was elected as it is ultimately situated in the abutment area of the mining panel. It runs parallel to the mining activities in 600E-600G and continues outside the mining area. This drift is located directly between the mining area and a 20 m wide regional support pillar, which allows additional investigations concerning the influence of the mining panel extension and backfill implementation on rupture phenomena in the support pillar.
3.3 Backfill stress measurements by flat jacks

In order to assess the influence of backfill on fracturing in rock mass or support pillars, information about stress built-up in the backfill body is essential. Especially the horizontal stress-built up is important as it can be linked to the lateral deformation of the pillar. For this research work, stress measurements in the backfill body at the Breitenau mine were conducted using 3 flat jacks, fixed on a steel frame (Error! Reference source not found.), to obtain the two horizontal stresses and the vertical stress built up in the backfill body.

![Steel frame for stress measurements](image1)

Figure 5 Steel frame for stress measurements

The frame was set up during the filling operations in situ on the backfill body at a height of 5 m (roof level of bottom drift). Afterwards the stope was filled until the floor level of the top drift (Error! Reference source not found.).

![Position of flat jacks in stope 600E at the Breitenau mine](image2)

Figure 6 Position of flat jacks in stope 600E at the Breitenau mine
Figure 7  Backfill stress measurements in stope 600E at Breitenau mine

Figure 7 shows the results from the stress measurements in stope 600E. So far the results do not show any exceptional increase in backfill stresses. This may change when the adjacent stope 600F is mined. Past deformation measurements have shown that mining of an adjacent stope tends to result in further lateral deformation of the pillar. Regarding horizontal stresses (Horizontal 1) it has to be mentioned that due to a technical problem with the measurement of the pressure cell, a drop in stresses was obtained at the second measurement point. At the 6th measurement point, the stress measurements of “Horizontal 1” can be regarded as reliable, as they increased up to the level of the second horizontal stresses (Horizontal 2) and run parallel to them.

3.4  Pillar deformation throughout whole mining cycle

Figure 8  Overview over extensometer measurements
Since 2009 multiple extensometer measurements have been conducted at the Breitenau mine in order to monitor the displacements in the pillars created by open stoping as well as in abutment pillars (Figure 8). These measurements, in combination with a detailed documentation of the mining activities throughout the mining cycle, offer the unique possibility to link the lateral deformation of support pillars to mining activities and backfill implementation. The position of the extensometers used in the following analysis is shown in Figure 8. In Figure 9 the extensometer displacement measurements for three pillars, namely pillar 600A on sublevel 1/2, pillar 600A on sublevel 2/3 and pillar 600 on sublevel 2/3 are presented.

The blue and red curves in Figure 9 represent the displacements in mm for the 5.5 m (blue) and for the 3 m (red) extensometer. The violet and pink marks indicate the time of sill pillar extraction (each cross stands for one blast) in stope 600A and 600B, whereas the green marks illustrate the time of backfill implementation in the stopes. The critical parameter is the time between completion of sill pillar extraction and backfill implementation. In this case, the time span between sill pillar extraction and backfill implementation in stope 600A was 2 months due to technical problems with the backfill system. This delay resulted in a steep rise in lateral deformation in the support pillar. After the completion of backfill implementation, the rate of lateral deformation slows down. No further increase in lateral displacement in the pillar was observed after backfill implementation in stope 600B. This deformation behaviour highlights the importance of immediate and quick backfill implementation like it was conducted in stope 600B. In this case backfill placement was executed 40 days after completion of sill pillar extraction (which is almost twice as fast as in 600A) combined with a filling completion after 16 days.

Two extensometers (light blue and light red curve) were implemented between backfill implementation in stope A and B to monitor the influence of backfill on displacements in the pillar. The results from these 4 extensometers clearly reveal the effect of backfill implementation on pillar deformation.

![Figure 9 Lateral deformation of pillar 600A TS1/2](image.png)
For the same pillar, between sublevel 2 and 3, similar results were obtained. This example shows even more clearly the influence of backfilling on lateral deformation in the pillar. Between complete sill pillar removal and backfill implementation in stope 600A most of the lateral deformation in the pillar occurred (45 days), which shows how critical time of backfill placement is. Filling operations in this stope lasted 128 days and directly after backfill implementation completion the lateral deformation of the pillar ceased and continued to stagnate around a constant deformation (Figure 10).

![Figure 10 Lateral deformation of pillar 600A TS2/3](image)

Regarding the abutment pillar 600 the same results for pillar deformation related to backfill placement could be obtained. The main displacement in the pillar could be observed during the time span between the completion of sill pillar extraction and backfill implementation (45 days). After backfilling started, the lateral displacements in the pillar ceased and remained constant thereafter (Figure 11).

![Figure 11 Lateral deformation of pillar 600 TS2/3](image)
3.5 Effects of increasing the number of stopes on abutment pillar 600H

For the evaluation of the regional stability of the mining panel in the Breitenau mine, extensometers were installed in abutment pillar 600H. So far the influence of the excavation of two stopes (600C and 600E) on the lateral deformation in the abutment pillar 600H which has a width of 20 m has been observed by means of the extensometer monitoring. To date the displacements add up to 5 mm. The advance in stoping activities can directly be linked to an increase in lateral deformation of the abutment pillar (Figure 12).

![Figure 12 Influence of stoping activities on abutment pillar deformation](image)

4 Numerical analysis

To evaluate the influence of stoping activities on the abutment pillar 600H, a numerical simulation of the mining panel 600 was made. The analysis assumes linear elastic rock mass behaviour which is aimed at obtaining approximate results of the stress changes in the abutment pillar 600H as a result of ongoing stoping operations. For simplification it is assumed that the stopes are not backfilled.

In the following, the results of stress changes in the abutment pillar as a result of stoping activities in stope 600F are presented. The simulation focuses on the vertical and horizontal stresses and the horizontal displacements and is computed by the software EXAMINE 2D provided by Rockscience. The parameters for the calculations are: density of overburden of 2700 kg/m³, depth of 900 m, a horizontal stress ratio of 0,5, a Young’s modulus of 18 GPa and a Poisson ratio of 0,25. Using these parameters a primary vertical stress of 24,3 MPa and primary horizontal stresses of 12,15 MPa were obtained.

Two different mining stages are analysed: panel 6 after mining of stope 600E and panel 6 after mining of stope 600F. The following illustrations show the results, which are given by the displacement and stress fields. Moreover the changes in the stresses and displacements resulting from mining activities in stope 600F are outlined along a measurement line. The measurement line is horizontal and placed at the location of the installed extensometers 1,20 m above the floor of the
An experimental study to investigate the interaction of backfill and rock mass

A. Moser, F. Wallner, H. Wagner, T. Ladinig

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Figure 13  Vertical stresses before stoping in 600F

As can be seen from Figures 13 and 14 the changes in vertical stress as a result of mining stope 600F are very small. Similar results have been obtained for the horizontal stress changes.

The analysis of horizontal displacements after mining of stope 600E and stope 600F shows more pronounced changes in horizontal displacements in the area of abutment pillar 600H. These are summarized in Figure 15.
The zone of lateral displacement in the abutment pillar 600H (around 6 mm in direction of the drift) shows an extension further into the abutment pillar. The zone of displacement around the stope which is adjacent to the drift 600H extends with the same magnitude in both mining steps. Along the measurement line in drift 600H (position of extensometers), lateral displacements of around 6mm were observed after stoping in 600F. The results of this initial analysis tend to support the displacement measurements shown in Figure 12.

![Figure 15 Elastic lateral deformation along measurement line](image)

5 Conclusion

The results of the first phase of the field studies in the deep mining section of the Breitenau magnesite mine can be summarized as follows:

- The field studies have demonstrated the capabilities of the new stoping system for the deep sections of Breitenau magnesite mine which is based on a system of 21 m high and 6 m wide stopes which are separated by 7 m wide and 21 m high stope pillars. To ensure stability of the slender pillars the stopes are backfilled.
- The application of cemented backfill in the 21 m high and 6 m wide stopes has very beneficial effects in the deformation behaviour of the 7 m wide and 21 m high stope pillars.
  - The most critical factor is the time span between sill pillar extraction completion and backfill placement.
  - The beneficial effects of backfill are noticeable even before the backfill has had time to cure. This raises questions concerning the strength and deformation properties of backfill. Stress built up in the backfill body corresponds to the dead weight of the fill column. Due to the fact that lateral pillar deformation is effectively arrested by the placement of the backfill no increase in lateral backfill stress has been observed so far.
  - Due to panel extension, fractures and lateral deformations have been observed in the panel abutments indicating regional stress transfer illustrating the need for control of stope panel dimensions.
  - Extensometer monitoring represents a simple, accurate and adequate measurement tool for lateral pillar deformation that can simply be read off by mining personnel.
References


An instrumentation project to investigate the response of a ground support system to stoping induced deformation

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Abstract

As a sub-project of the industry-sponsored Ground Support Systems Optimisation Project being delivered by the Australian Centre for Geomechanics (University of Western Australia), an instrumented trial has been carried out at George Fisher Mine (part of Glencore’s Mount Isa Mines operation) Mount Isa, Queensland, Australia. The aim of the trial was firstly to measure the response of a ground support system to mining related excavation in close proximity and secondly to replicate that response by means of a numerical model. This paper deals with the trial site, instrumentation and data acquired. Displacement measurements were taken using a tape extensometer and also by means of photogrammetry. An array of SMART cables was used to determine support load distributions and extents of the deformation zone. Fibrecrete liner load was measured by means of vibrating wire strain gauges embedded in the liner. Observation holes were used to verify the extent of the fracture zone around the trial site excavations. Displacements, loads and strains were tracked by manual and automated data retrieval methods and are presented in the paper. A discussion of the results and their implication for numerical modelling of ground support is presented.

1 Introduction

The Australian Centre for Geomechanics commenced the industry funded research project “Ground Support Systems Optimisation” (GSSO) in late 2013. The project aim is to explore whether it is possible to optimise ground support systems to improve mine safety and economics. The project comprises three sub-projects, namely: 1) probabilistic ground support design; 2) the use of numerical modelling for ground support design; and 3) benchmarking of current ground support practices. This paper focusses on sub-project 2 and features an underground instrumentation project designed for the purposes of calibrating a numerical model.

The trial has been implemented at a GSSO sponsor site, namely the George Fisher Mine, located approximately 22km north of Mount Isa, Queensland. A locality map is given in Figure 1.

George Fisher Mine extracts lead, zinc and silver from deformed and metamorphosed Proterozoic stratiform deposits hosted by dolomitic and carbonaceous sediments of the Urquhart Shale. The ore is mined by a combination of benching and transverse open stoping.

2 Objective

The objective of the instrumentation project was to gather rockmass and ground support response data for the purpose of calibrating a numerical model. The requirements for the instrumentation sites were as follows;

- Simple geology and geometry preferable (to minimise variables in the modelling process).
- Potential for rockmass deformation (high stress / low strength environment).
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- Mine-by of nearby stopes to induce load / relaxation cycles.

The monitoring methods used were selected with the goal of providing parameters easily measurable in the field as well as in a numerical model, as follows;

- Closure measurement (Tape Extensometer) – tried and tested method of sidewall deformation measurement intended as a backup check of other deformation measurement techniques (photogrammetry and laser). Results are directly comparable with numerical modelling deformation predictions.

- Closure measurement (photogrammetry) – modern technique for creating a 3D georeferenced model of the excavation skin, using a mosaic of overlapping high-resolution photographs. Successive scans can be compared with each other to gain a 3D profile of the excavation deformation, for comparison with numerical model predictions.

- Shotcrete liner strain – vibrating wire strain gauges embedded in the liner, directly measuring the circumferential strain within the liner shell.

- Cablebolt load – SMART cables (Hyett *et al*, 1997) installed in a fan around the excavation perimeter, to directly measure strains in the cable element and indirectly indicate the extent of the rockmass yield zone.

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![Map of Australia showing the locality of the George Fisher Mine in NW Queensland](image)

**Figure 1** Map of Australia showing the locality of the George Fisher Mine in NW Queensland

### 3 Mining environment

Two sites were made available for the installations, both on the 17 level, approximately 1060m below surface. In Figure 2, the mine layout is conceptually shown, with the instrumentation sites circled blue in the inset.
At the time of the installation, the north and south sites were both outside the stoping fronts, in the stress abutment zone. The mining schedule at the time of installation dictated that the mining front would progress beyond both sites during the project timeframe.

Figure 2  Location of the instrumentation sites (ringed blue in the inset) on the 17 Level at George Fisher Mine (viewed from the footwall). Development (green) and stoping (yellow) are as at the commencement of the installations in late 2014 and early 2015

The development layout and profile for each site are shown in Figure 3. Existing rockbolt support consisted of 2.1m resin encapsulated thread bar on an approximately 1.0m x 1.0m pattern. Surface support consisted of weldmesh (and partial fibrecrete at north site).

Transverse open stopes of varying dimension were to be extracted in a primary/secondary sequence using pastefill for primary stope support. Level spacing is 30m, stope width (along strike) is 15m and length (transverse) varies up to approximately 40m, depending on orebody dimension. Stopes are sequenced in a chevron pattern to ensure no pillar remnants are formed during the extraction process. The conceptual stoping sequence is as shown in Figure 4.
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North Site – plan view

South Site – plan view

Sectional view

Sectional view

Figure 3  Development layout and drive profiles for the north and south instrumentation sites (existing rockbolts indicated in red)
4 Geotechnical environment

Both instrumentation sites are located in the footwall of the orebody, in a closely bedded shale and siltstone unit known as the Urquhart Formation (Figure 5). The bedding at the instrumentation sites dips at ~55-60° to the west.

Intact rock strength is variable and anisotropic, with a mean value of 90MPa and an Anisotropy Factor of 3.0 (Vakili, Albrecht and Sandy, 2014).

The in-situ stress field has been determined by hollow inclusion (HI) cell techniques and is given in Table 1.

Table 1  

<table>
<thead>
<tr>
<th>Principal Stress Component</th>
<th>Magnitude (MPa)</th>
<th>Orientation (dip/azimuth)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Maximum</td>
<td>46.2</td>
<td>24°/062°</td>
</tr>
<tr>
<td>Intermediate</td>
<td>38.5</td>
<td>04°/153°</td>
</tr>
<tr>
<td>Minor</td>
<td>27.5</td>
<td>66°/256°</td>
</tr>
</tbody>
</table>
5 Instrumentation

The monitoring methods used are discussed as follows;

*Tape extensometer* for sidewall closure (Figure 6). Closure pins were set up at the locations shown in Figure 7 and read at weekly intervals.

*Photogrammetry* for tunnel skin deformation. The ADAM Technology (2016) system was selected as the method used for underground scans and image processing. The field setup is as shown in Figure 8. Scans were taken each time a significant event occurred in the stoping sequence or a jump was noted in the sidewall closure readings.

*SMART cables* for support load and rockmass deformation were supplied by Mine Design Technologies - Australia (Hyett, AJ, et al 1997). Five 8m cables were installed in a fan at each site, as illustrated in Figure 9. The cables were installed with the readout head at the toe of the hole to prevent damage and with the anchor nodes positioned as indicated in Figures 17 to 26. Initially, readings were carried out manually on a weekly basis, but ultimately they were connected into the mine’s existing Newtrax Recording System, also provided by Mine Design Technologies (Australia) allowing automated readings to be taken at a specified frequency.

*Vibrating wire strain gauges* for fibrecrete liner deformation. Supplied by Encardio, these instruments consist of a central vibrating wire element attached to segments of rebar as shown in the installation in Figure 10. The instruments were attached to the existing weldmesh then shotcreted in place. Four instruments were installed at each site as also indicated in Figure 9.
Figure 6  Tape extensometer used for sidewall closure measurement (photograph provided by Mining Innovation Rehabilitation and Applied Research Corporation)

Figure 7  Locations of closure pins on sidewall
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Figure 8 Photogrammetry field setup, consisting of tripod mounted digital camera and floodlighting

Figure 9 Fan arrangement of SMART cable instrumentation. Approximate positions of the shotcrete straingauges are also shown in red.
Figure 10  Vibrating wire shotcrete strain gauge attached to weldmesh and ready for shotcreting. The vibrating wire element (blue) is oriented such that circumferential strains in the liner are measured, perpendicular to the tunnel axis

6  Results and discussion

Closure measurement results are shown in Figures 11 and 12, for the North and South sites respectively. Both sites show a steady increase in closure over time, the north site at approximately 0.5mm per month and the south site at approximately 0.2mm per month. Total closure for the north site averages around 7mm and for the south site 2.5mm. The linear increase in closure evident in Figures 11 and 12 could be indicative of a creep component of deformation.

Tunnel scans and profiles obtained by Adamtech photogrammetry surveys for the north site are shown in Figures 13 and 14. An equivalent sidewall closure of approximately 2cm is indicated, significantly higher than the closure measurements. It was found that significant variability in the results could be obtained depending on how the control points were specified. Further work on the control point survey methodology is required.

Shotcrete strain data results are presented in Figures 15 and 16, for the north and south sites respectively. Circumferential strain is generally compressive (negative strain), consistent with a general closure of the tunnel profile (shortening of tunnel perimeter).

SMART cable data is presented in Figures 17 to 24, for instruments which showed realistic results (sidewall cables). The other instruments do not indicate realistic looking data, due either to a) problems with installation or 2) undetectable rockmass movement. Displacement vs distance from toe is shown in Figures 17, 19, 21, and 23. The toe readout unit represents a ‘static’ point in the rockmass with cable stretch measured relative to this point. The inter-nodal loads calculated from the displacements are given in Figures 18, 20, 22 and 24.

Comparing the SMART cable displacements for the hangingwall and footwall (sidewall) cables, with the closure data gives mixed results. For the north site, the combined displacements at the cable collars totals approximately 8.5mm which compares very well with the extensometer measured closure range of 8-12mm. For the south site the cables indicate a closure of approximately 4.5mm, versus the extensometer
measured closure range of 3-3.5mm. The south site SMART cable installations were more challenging than the north (Broadus Jeffcoat-Sacco, pers comm), which could potentially have resulted in less reliable readings from the south site instruments.

The displacement profiles indicated by the SMART cables give an indication of the extent of the fracture (yield) zone surrounding the excavation at that location. From Figures 17, 19, 21, and 23 it can be seen that the depth of this fracture zone ranges from 1.5m to 2.5m, with the maximum being recorded by the hangingwall gradeline cable at the north site.

![Figure 11 Sidewall closure measurement results for the North Site. Data sets are coloured according to the closure pin pair indicated in the legend (one pin in each opposite wall as per Figure 7)](image1)

![Figure 12 Sidewall closure measurement results for the South Site. Data sets are coloured according to the closure pin pair indicated in the legend (one pin in each opposite wall as per Figure 7)](image2)
Figure 13  Superimposed photogrammetry scans (red: May2015, blue: November2014) indicating areas of the north site which have converged

Figure 14  Cross-section through the north site showing the relative deformation between November 2014 (blue) and May 2015 (red)
Figure 15  Shotcrete strain measurement results for the North Site

Figure 16  Shotcrete strain measurement results for the South Site
Figure 17  SMART cable displacement vs distance from sidewall, for hangingwall gradeline cable, north site

Figure 18  SMART cable load vs distance from sidewall, for hangingwall gradeline cable, north site

Figure 19  SMART cable displacement vs distance from sidewall, for footwall gradeline cable, north site

Figure 20  SMART cable load vs distance from sidewall, for footwall gradeline cable, north site

Figure 21  SMART cable displacement vs distance from sidewall, for hangingwall gradeline cable, south site

Figure 22  SMART cable load vs distance from sidewall, for hangingwall gradeline cable, south site
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8 Conclusions

The monitoring trial has been successful in delivering a dataset of sufficient quality to be used in calibrating a numerical model. Key performance elements of the system have been captured, namely tunnel deformation, cable support load and liner strain. The data obtained suggests that the system is behaving (conceptually) as would be anticipated.

Stopping in the vicinity of the north trial site during 2016 is anticipated to generate significantly more displacement than has been recorded to date, i.e. the results obtained to date are at an early stage.

Acknowledgements

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An optical sensor for capturing the three-dimensional bending of bolts

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Abstract
Capturing the three-dimensional load distribution along fully grouted bolts has been inherently difficult to accomplish with conventional strain techniques. A general lack of spatial resolution of strain measurements along instrumented bolts has resulted in localized loading conditions, such as shear and bending, being misinterpreted and, perhaps, even omitted. The historical development of such sensing techniques under controlled laboratory conditions, as well as field trails, has severely limited constraints in terms of monitoring behaviour as sensors are often required to be positioned in alignment with significant loading features (locations of which may not be known a priori). In addressing such limitations, a novel monitoring solution is presented whereby a distributed optical strain sensing technique with sub-centimeter spatial resolution is employed. The technique considers using one optical sensor to monitor three lengths of a bolt at 120 degrees from one another. The results from a symmetric bending, combined axial and bending load, and double shear configuration test are presented in order to demonstrate the capability of the monitoring solution to: 1. Resolve strain into axial and lateral (i.e. bending) components, 2. Determine the maximum magnitude of lateral strain, and 3. Derive the direction of the lateral strain vector.

1 Introduction
Fully grouted rock bolting is an extensively employed technique utilized in order to improve the stability and maintain the load bearing capacity of the ground surrounding an excavation in underground projects. In comparison to end anchored systems in which the bolt is actively loaded under uniform tension, the loading profile along a passive, fully grouted bolt is potentially much more complicated. In an ideal ground mass the surrounding ground movement will be coaxial with the bolt, initiating the support response in a purely axial fashion. In this manner the ground movements are assumed to be continuous and distributed over the length of a bolt as in the early work of Freeman (1978). However, ground movements may also be dominated by a number of discrete (i.e. localized) occurrences which may be created during the excavation process or pre-exist in the structure of the surrounding ground mass (Bjornt & Stephansson, 1983, Hyett et al. 1996, Li & Stillborg, 1999). In most cases, the movement of such discontinuous ground will not be aligned with the bolt and can also take on the form of shear displacement where the bolt will act as a dowel. Li (2010) has discussed such behaviour based on the observations of failed bolts displaying permanent shearing (and bending) that have been exposed after a fall of ground (Figure 1). Accordingly, rock bolts may be subjected to a combination of axial and shear loading throughout its support serviceability life which may take on a transition from distributed to localized loading. This ultimately results in a highly variable loading profile along the length of a fully grouted bolt. The support contribution under such conditions has been extensively studied both analytically and numerically by many researchers (e.g. Spang & Egger, 1990, Ferrero, 1995, Grasselli, 2005); however, the complexity associated with localized loading features, such as shearing, has been inherently difficult to capture with current monitoring techniques. This is primarily attributed to three-dimensional complexities (i.e. the location, magnitude, and orientation) of load distribution along the bolt, which are often difficult to predict beforehand due to geological uncertainties, installation uncertainties, etc. In this regard, a new optical bolt sensing solution is
presented with the capacity to capture the inherent behaviours of fully grouted rock bolts and to resolve them at a sub-centimeter scale along the length of a bolt. This research has been performed within the scope of demonstrating the competency of the proposed solution and as a required predecessor before advancing the current knowledge state of shearing along bolt support.

Figure 1  Permanently deformed rebar element after a rock burst event. Lateral deformation of the bolt is clearly evident (Image courtesy of Brad Simser, Glencore).

2  Bolt instrumentation

Anchor pull-out tests are the most commonly performed assessments of fully grouted rock bolts; however, they may not provide an accurate representation of the loading conditions of a passive bolt over its support life. This may also be the case for assessments with a methodology that is extrinsic to the bolt or monitors the bolt at the excavation periphery (e.g. pressure washers). In this regard, this section provides an overview of existing sensing techniques that have been implemented in an effort to monitoring the distribution of load along the length of a bolt element. The section also presents a new proposed technique for capturing the three-dimensional performance of bolts.

2.1  Previous sensing techniques

Early work completed by Farmer (1975) and Freeman (1978) considered surface mounting an array of foil-resistive strain gauges to a machined surface along the length of rebar specimens. The distribution of strain along the length of the support element could, therefore, be determined through the interpolation of discrete measurements provided by each strain gauge. Serbousek & Signer (1987) further developed this solution by positioning similar strain gauges within a pair of diametrically opposed grooves machined out along the length of rebar specimens. This provided additional protection to the instrumentation and also allowed the strain distribution to be separated into axial and bending components through a comparison of the measured strain along opposing sides of the bolt. However, the short base-length of each strain gauge results in large sections of the bolt being left unmonitored (i.e. the space between gauges). Consequently, localized loading features along the specimens are prone to being underestimated and possibly omitted (e.g. localized shearing). Spearing et al. (2013) presented a similar technique where the short base-length resistive strain gauges were replaced with long base-length inductive strain gauges (commonly 200mm to 500mm gauge lengths). This allowed for the entire length of a given bolt element to be monitored by discretizing it into measurement “zones” as opposed to measurement points. Yet, similar to the traditional strain gauges, the long base-length gauges may result in peak load being underestimated if loading is not uniform across the gauge length. The technique cannot readily distinguish between single or multiple loading features and the long base-length will essentially average out any localized shearing.
The spatial limitations of electrical strain techniques has led many researchers to consider innovative sensing techniques such as fiber optic sensing. Perhaps the most intriguing aspect of optical sensing is the potential to use one optical fiber as both the lead and transducer for an array of measurements (i.e. a continuous or sub-continuous measurement sensor). Schroeck et al. (2000) has discussed the use of multiplexed fiber Bragg gratings (FBG) to monitor the strain along bolts. This technique is optically analogous to the short base-length resistive strain gauges in regards to providing a single measurement point per Bragg grating. Ultimately, the FBG solution will be limited by the number of Bragg gratings that can be inscribed per sensor (effectively 10) and therefore, will experience similar spatial resolution issues as described for the electrical techniques. However, a promising solution to spatial resolution limitations can be realized through the application of distributed optical strain sensing (DOS), whereby strain is monitored continuously along the length of a standard single mode optical fiber. Iten & Puzrin (2010) and Hyett et al. (2013) have demonstrated examples were a single optical fiber has been embedded within a pair of diametrically opposed grooves machined out along a ground anchor and rebar specimen, respectively. Of particular interest is the optical frequency domain reflectometer (OFDR) technique utilized by Hyett et al. (2013) which was demonstrated to have the capacity to capture strain at a spatial resolution of 1.25mm (i.e. 1.25mm between strain measurements) along the entirety of the bolt’s length and, for the first time, capture the strain profile along an entire rebar specimen subjected to shear. Yet, a major limitation of this solution, and the aforementioned techniques using two sensing lengths, is the inability to resolve the maximum magnitude of strain, except for the limiting case when the sensing lengths are directly in plane with bending, lateral loading, or localized shearing. In most situations, this can be controlled in the laboratory setting, but in situ may substantially hinder the competency of the solution. This limitation and issue is explained in more detail in Section 3.1 of this paper.

2.1 Proposed solution

From the success of Hyett et al. (2013) and Forbes (2015) it was decided to develop a new bolt sensing solution with the capacity to capture the three-dimensional behaviour of rock bolts. The proposed solution considers the OFDR technology as the sensor, but unlike previous efforts, is arranged such that one optical sensor monitors three machined out lengths along the bolt at 120 degrees from each other. A comparison of the existing ‘two-groove’ solution and the proposed ‘three-groove’ is visualized in Figure 2.

![Figure 2](sensor_configurations.png)

Figure 2  Sensor Configurations: Left – Two diametrically opposed machined lengths. Right – Three machined lengths spaced 120 degrees from each other

Positioning the optical sensor in such an arrangement allows for the maximum strain magnitude along with its true orientation to be calculated along the length of the bolt according to Equation 1. This can further be broken down into axial and lateral strain components, the first and second term in Equation 1, respectively. The orientation of bending, lateral load, or shear (i.e. the lateral strain vector) can be resolved with respect to the first sensing length (i.e. the first machined out groove along the bolt at 0 degrees, Figure 2) according to Equation 2. It is important to note that this solution is performed at the same sub-centimeter spatial resolution along the entirety of the bolt. In this regard, both the load orientation and location(s) along the bolt does need not to be known prior to installing the sensor, making it an ideal solution for resolving complex ground conditions.
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where:

\[ \varepsilon_{\text{Bolt}} = \left( \frac{\varepsilon_1 + \varepsilon_2 + \varepsilon_3}{3} \right) \pm \sqrt{\frac{2}{3}} \left( (\varepsilon_1 - \varepsilon_2)^2 + (\varepsilon_2 - \varepsilon_3)^2 + (\varepsilon_3 - \varepsilon_1)^2 \right) \]

where:

\[ \varepsilon_{\text{Bolt}} \] = The maximum/minimum value of strain along the bolt periphery

\[ \varepsilon_i \] = The strain captured along the specified sensing length (i.e. at 0°, 120°, or 240°)

2

where:

\[ \theta = \frac{TAN^{-1}}{2} \left( \frac{\sqrt{3} (\varepsilon_1 - \varepsilon_2)}{2 \varepsilon_1 - \varepsilon_2 - \varepsilon_3} \right) \]

where:

\[ \theta \] = The orientation in degrees of load/displacement direction with respect to the first sensing length (i.e. the sensor at 0°)

\[ \varepsilon_i \] = The strain captured along the specified sensing length (i.e. at 0°, 120°, or 240°)

3 Initial validation experiments

Controlled laboratory experiments were initially performed in order to assess the optical or OFDR solution. This included: a) a symmetric bending load test and, b) a combined axial and bending load test. This was determined to provide the most controlled conditions for verifying the proficiency of the solution to separate components of strain prior to conducting more complex shear loading experiment.

3.1 Symmetric bending

Prior to conducting a symmetric bending test with the new OFDR sensing technique (i.e. three sensing lengths), the experiment was first conducted using an optically instrumented rebar specimen with two diametrically opposed sensing lengths (Figure 3). The specimen was positioned on two roller supports approximately 0.80m apart and load was applied in plane and at the center position between the two supports.

Figure 3 Symmetric bending configuration of an optically instrumented rebar with two machined lengths. Red circles indicate the position of roller supports. The respective position of the optical sensor is also displayed (note: this corresponds to a single optical sensor)

Figure 4 presents the strain profile along the optical sensor (i.e. the two grooves along the rebar) measured during two tests under the specified loading configuration. The left plot presents results from a test when the two sensing lengths were oriented in plane with the direction of applied load. The right plot presents results from a test when the two sensing lengths were oriented approximately orthogonally with the direction of applied loading (i.e. in proximity with the neutral axis of the specimen). The strain profiles along opposing sides of the rebar are observed to essentially mirror one another: a positive (i.e. tensile) strain profile along the first sensing length: 0.70m-1.50m, and a corresponding negative (i.e. compressive) strain profile along the second sensing length: 2.00m-2.80m. This implies the strain is fully constituent of
bending load. The zero magnitude of strain measured between the opposing lengths (1.50m to 2.00m) corresponds to the position where the optical sensor has been looped.

As expected, the orientation of the sensing lengths in comparison to the direction of bending controls the magnitude of strain that is captured. This implies that the given technique is prone to misinterpretation of bending. In most cases, the measured strain value will underestimate the extent of bending as the full magnitude will only be measured when the two sensing lengths are in plane with the direction of bending. At all other orientations the measured magnitude of strain will be a lesser value and potentially zero if situated along the neutral axis. This is a concern if the direction of bending, lateral loading, shearing, etc., is not known beforehand (or is redistributed with excavation development) as is often the case in situ.

The misinterpretation of bending is avoided with the addition of a sensing length as per the proposed technique. To demonstrate this, a similar bending test was conducted using a rebar with three sensing lengths positioned 120 degrees from one another (Figure 5).

An example plot of the strain captured along the length of the optical sensor (i.e. the three grooves along the rebar) is presented in the leftward plot of Figure 6. The rightward plot of Figure 6 displays the true maximum magnitude of strain along the rebar determined using Equation 1.
3.2 Combined axial and bending

A combined axial and lateral (i.e. bending) load experiment was devised in order to demonstrate the capability of the sensing technique to resolve components of axial and lateral strain along the rebar. The loading involved initially applying an axial load (i.e. tension) of two tonnes to the rebar. Thereafter while axial load was held, a lateral load was applied at an asymmetric position (generalized case). The apparatus used in this experiment is presented in Figure 7.
Figure 7  Combined tension and lateral loading of an optical instrumented rebar. A 20 ton cylinder is used to apply axial load to the rebar. A turnbuckle is used to apply lateral load to the rebar. Spherical washers and faceplates were used to minimize bending components during tensioning of the rebar

Figure 8 displays the strain profiles captured along the optical sensor, and therefore, the three sensing lengths along the rebar. Sharp drops in the strain profiles at 1.70m and 3.20m correspond to the locations where the optical sensor has been looped, as previously discussed. This allows for the three sensing lengths to be easily distinguished from each other. As discussed for the symmetric bending test (Section 3.1), the orientation of the sensing length in comparison to the direction of applied lateral load will control the magnitude of strain that is measured. This is immediately apparent when comparing the shape of the strain profiles captured along each sensing length. As noted by Hyett et al. (2013) and Forbes et al. (2015), applying a purely axial load is a difficult, if not impossible, task to accomplish. Any slight imperfections in the alignment of the loading apparatus, initial seating of washers, imperfection of the rebar, etc., will result in a component of bending. This is discernible in the strain profiles during the initial axial loading of the rebar (i.e. at 1 tonne and 2 tonnes). The profiles are observed to have either convex or concave like shape (indicative of bending) rather than a uniform strain level along each length. The succeeding application of lateral load is then observed to either increase or decrease the magnitude of strain along the bolt length depending on the orientation of the sensing length. Using Equation 1, the strain along the bolt can be separated into axial and lateral components. The resultant axial and lateral profiles are presented in Figure 9.
Figure 8  Combined tension and lateral load experiment: Strain profiles captured along the optical transducer. One sensor is used to monitor three lengths of a bolt subjected to axial and lateral loading. The first sensing length (approximately 0.10m to 1.60m) was oriented in plane with the applied lateral load.

Referring to Figure 9, the leftward plot indicates that, as expected, the magnitude of axial strain is consistent along the bolt. This is observed to increase from 135με at one tonne to 275με at two tonnes, but more importantly, thereafter, it is seems to be unaffected by the application of lateral loads. This demonstrates that the sensing technique can successfully separate the axial component of strain along the bolt in addition to determining the maximum lateral component of strain along the bolt as shown in the rightward plot. Unlike the resolved symmetric bending example displayed in Figure 5, non-zero bending strains are prevalent at both ends of the bolt. The difference between these two plots can be attributed to the unique end conditions of each test. In contrast to the symmetric bending test, which is ultimately a three-point-bending test for which the ends of the bolt are free, in the combined loading test the ends of the bar are constrained through the end holding devices associated with the application of axial load. In this regard, the resolved lateral strain profile for the combined loading test agrees well with the solution of a 'built-in-beam' (Figure 10).
4 Shear block experiments

A double shear load experiment similar to that presented by Hyett et al. (2013) was performed in order to demonstrate the effectiveness of the proposed solution. This test considered centering and cement grouting (0.40 w/c ratio Portland cement) an optically instrumented rebar specimen within a pre-cast 50mm borehole running through three individual concrete blocks (40MPa UCS). A thin (3mm) vertical plane was created between the concrete blocks by positioning a nylon sheet in between blocks during casting. This was performed in order to promote ‘pure’ shear displacement between the blocks (i.e. reduce uncertainties arising from dilation and frictional resistance at the concrete interface). During grouting the rebar was best oriented such that the first sensing length was best aligned with the vertical displacement of the central block. The double shear test configuration is displayed in Figure 11.
A servo-controlled 500kN loading frame was used in conducting this experiment. This allowed for the actuator load and stroke to be controlled and monitored at all times during the testing. A spring loaded linear-variable-displacement-transducer (LVDT) was also attached to the central block to provide a redundancy in displacement measurements. Figure 12 displays the strain profiles captured along the entire length of the optical sensor (i.e. the three grooves along the rebar) at various central block displacements as well as a description of the sensor orientation.

![Figure 12](image)

**Figure 12** Double shear loading: Left – Orientation of sensing lengths relative to the concrete blocks (although this was skewed during grouting). Right – Strain profiles captured along the optical sensor at various central block displacements

At each discontinuity (i.e. shear plane) a distinct shear couplet is measured. This corresponds to a compressive strain on one side of the discontinuity which is accompanied by a tensile strain on the opposing side. The shear couplet is a combination of both bending (i.e. lateral) and axial strain and has been similarly discussed as an ‘S-bend’ (McHugh & Signer 1999) and as a ‘plastic hinge’ (Grasselli 2005); although, the sharp strain gradient across the shear plane has not be measured to the same extent with techniques other than the OFDR. As with the bending experiments, the impact of the sensor orientation on measured strain magnitude is obvious.

Figure 13 presents the resolved axial and lateral strain component calculated using Equation 1 for a central block displacement of 2.0mm. Bending is the most evident component of strain observed as a result of the shear loading; however, the component of axial strain is not insignificant. Under the configuration previously described the axial strain is observed to propagate almost half a meter away from the shear planes and correspondingly overlaps to form a zone of tension within the central block. This is an interesting outcome as under non-idealized conditions a more prominent dilation component across the shearing joint would be expected to induce further axial loading of the bolt (Egger & Zabuski 1991).

Figure 14 displays a comparison of the strain profile captured along each sensing length at a central block displacement of 2.0mm as well as the resolved orientation of the lateral strain vector according to Equation 2. In the rightward plot a 180 degree re-orientation of load is observed across the shear plane which is expected in accordance with the inflection point of the bolt. This can be compared to the rightward plot of Figure 12 where a 180 degree flip coincides with minima positions on the lateral strain plot. The bolt was resolved to be oriented clockwise approximately 10 degrees from an alignment of the first sensing length with the central block displacement (i.e. installation imperfection). This provides an explanation to the lower strain magnitude captured along the third sensing length, which evidently was oriented towards the neutral axis (Figure 12).
Figure 13 Double shear loading: Left – Resolved axial strain profiles along the grouted rebar. Right – Resolved lateral strain profiles along the grouted rebar (note: this is an absolute magnitude)

Figure 14 Double shear loading: Left – Comparison of strain captured along the three machined lengths at a central block displacement of 2.0mm. Middle – Resolved orientation of sensing lengths with respect to the concrete blocks. Right – Resolved orientation of loading direction with respect to the first sensing length

5 Conclusion

This research has presented a novel sensing technique with the capability of capturing complex loading profiles, such as shearing, along the length of a bolt support element. The strain profiles measured throughout a laboratory testing scheme have demonstrated the technique’s potential to resolve captured strain at sub-centimeter increments along the bolt into separate axial and lateral components. Derivation
of the principle loading direction as well as the true maximum magnitude of strain is also permitted with the sensing solution. In this regard the sensing technique does not require a prior knowledge of significant loading location(s) or direction in order to capture an accurate representation of the bolt loading profile.

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Building a structural model for assessing potential pit slope failure – a case of a Norwegian mine

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Abstract

Joints and faults are inherent part of the rock mass. In the vast majority of the mining slopes, discontinuity structures play an important role in slope stability and may trigger potential slope failure. The most important step in understanding the slope failure mechanism is to have a reliable model, which shows how all the discontinuity sets are constituted in the rock mass and how they are interacting to each other. On the other hand, building a fracture model is not a straightforward process, since it needs to combine discontinuity information from different sources, such as detailed slope mapping, borehole logging data and remote sensing technologies. The final objective is to have a consistent and trustworthy fracture model that shows the main lineaments, orientations and characteristics of the different discontinuity sets in a three-dimensional approach. The model will be also the baseline for the future elaboration of a geotechnical model.

This paper will attempt to develop a comprehensive structural model of a particular area in a Norwegian open pit mine, the biggest in Norway with respect to its depth and area of coverage. The process involves the analysis of different sources of data in order to correlate all these information into useful evidence about the characteristics of the main discontinuities in the area in terms of dip and dip direction. Data from extensive drilling, information gathered from remote sensing and discontinuity results obtained from extensive field mapping by authors and other scholars in the past will be exploited for this purpose. It is expected that the end result will be able to assess the credibility and validity of different approaches in mapping the discontinuities and also try to figure out potential general pit slope failure modes.

1 Introduction

Building a fracture model of a mine site is not a straightforward task. It often involves compiling information from many different sources such as: boreholes, field mapping, remote sensing, aerial images, geophysical investigations and others. Villaescusa and Brown (1992) states that a complete two-dimensional description of joint set characteristics are in many occasion difficult to establish due to limited size of rock exposures and access problems while field mapping. In the last 10 years, the development of remote sensing technologies has however helped to map areas with difficult access. The results of the application of photogrammetry into the investigation of rock discontinuities orientation shows that no significant errors are present if the process is done in the correct way (Lee et al, 2000).

On the other hand, the construction of a structural model is mainly based in the definition of the main joint sets and description of its persistence and frequency. Hudson and Priest (1983), Einstein (1983), and Zhang and Einstein (2000) have studied the intensity and frequency of joint persistence in the rock mass and its effect in the rock slope stability. In this field and with the actual widespread imagery of aerial pictures it is not hard to define the orientation and spacing of the main lineaments in a certain area of interest due to the good exposure of the rock mass. Nevertheless, this does not provide any information about the dip of the joint sets linked to these major fault planes.
The development of the structural model is the first stage in building a geotechnical model of a mine site. The value of an early geotechnical assessment has been described as the need to establish an appropriate level of geotechnical risk balanced against other key drivers at each stage of the mine planning process (Hanson et al, 2005). In addition, a geotechnical model could be helpful to provide production optimizations in the mine to mill value chain (Bye, 2006).

In a similar line, the Rock Quality Designation (RQD) is a key input parameter in the classification of the rock mass. Several authors like Haines (1991), Bye and Bell (2001), Pantelidis (2009), and Hormazábal (2009) have tried to find a suitable way to correlate the rock mass classification systems with the stability of a slope. In the same way, it is possible to use values of the RQD in estimating deformation modulus of the rock mass (Zhang and Einstein, 2004).

The main objective of this article is to assess the credibility and validity of three different approaches for mapping discontinuities consisting remote sensing, acoustical inspection of boreholes, and field mapping of tunnels and mapping of several geotechnical windows on the different benches of the slope itself. In addition, a bottom-up approach is employed to identify the most important and influencing lineaments (in terms of slope stability) in a selected sector of the open pit, based on the orientation and characteristics of the locally identified joint sets. In addition, a statistical estimation of a correlation between RQD and fractures per meter is presented. Finally, an analysis of the main joints and their interaction with the slope face is described. The obtained results are intended to be expanded in other zones of the pit to establish a complete geotechnical model of the mine, as shown in Figure 1.

![Figure 1](image.png)

**Figure 1** The open pit topographic map. Hanging wall is located in the SW while footwall is to the NE. The top of the footwall is here at level ~250 masl, and the bottom is at level ~40 masl

### 2 The project case

The open pit mine has been in operation since 1960. In the orebody, about one third of the rock consists of ilmenite. It is located inside in the Åna–Sira anorthosite, and it consists mainly of an ilmenite-rich norite, that has previously been interpreted as injected in a crystal mush state in a weakness zone of the enclosing anorthosite. This emplacement mechanism has produced a faint orientation in the ore due to the flow of
The mush (Diot et al, 2001). The mine is the largest titanium deposit in Europe and probably also the largest deposit in production in the world today. It is also one of the major titanium hard-rock deposits known today. Figure 1 above gives an overview of the topography of the mine as it was by April 2015.

The first pass of the walls of the open pit have an initial single bench height of 15 meters, which is doubled in the second pass to have an overall height of 30 meters. Therefore, the overall slope angle is about 55°. The open pit has a length of about 2.8 km, while the current depth is close to 240 meters. The width of the pit varies from 400 meters to 600 meters (Botsialas and Mass, 2014). The current mine plan is to extend the total depth to around 300 meters.

2.1 Geological setup

2.1.1 Regional geology

The Rogaland igneous complex, covering about 1200 square kilometers, dominates the geology in southern Rogaland. The complex consists of anorthositic, noritic and mangeritic intrusions, and jotunitic to charnockitic migmatites. It was formed in late Proterozoic time (930 Ma) and is surrounded by Precambrian gneisses (Duchesne, 2003). It is well known for its four large anorthosite massifs: the Egersund-Ogna, Håland-Helleren, Hellern and Åna-Sira massif, the last one being where the mine is located (Figure 2).

Figure 2 The Rogaland anorthosite province (taken from Karlsen, 1997)

The noritic intrusions occurred at a late stage of the genesis, and appear as several smaller intrusive bodies located in the south-eastern part of the province. Some of these norites, like the deposit where the mine is located, contains the richest ilmenite-bearing deposit known in the world (Marker et al, 2003).

2.1.2 The Åna-Sira massif

The Åna-Sira massif covers more than 100 square kilometres in the Sokndal region and consists mainly of anorthosite. It is often considered the most homogenous of the anorthosites in the Rogaland igneous
complex. The massif hosts significant resources of ilmenite-rich norite, like the Storgangen and the deposit where the mine is located. It is described as a quite fresh and unaltered anorthosite with medium coarse grained with mega crystalline texture and a grey/violet/brown colouration. The region has been subject to some hydrothermal alteration, which can be seen as a white-grey anorthosite often with shades of pink and green, and with a fine-grained texture. Several small zones of alteration are present (Karlsen, 1997). The massif is encapsulated by the Bjerkreim-Sokndal Lopolith that mainly contains noritic rocks. The anorthosite is cut by several mangerite, noritic and jotunitic dikes, a few bodies of ilmenite norite, a noritic layered intrusion (the Bøstølen intrusion) and a swarm of younger diabase dikes. Megacrystic Ca-poor pyroxene appear sporadically. The most common mafic minerals are pyroxene, ilmenite, biotite, amphibole and chlorite (Marker et al, 2003).

2.1.3 Open pit geology

The ilmenite deposit is a world-class Fe-Ti mineralization that consists of an ilmenite rich norite lens-shaped body (> 400m x 2700m), which crops out in the central part of the Åna-Sira anorthosite. The deposit is known to at least 60 meters below sea level. At both ends it extends into mangeritic dikes, about 5-10 m thick, which are stretching out to the north-west and to the south-east directions for several kilometres.

The structure of the ore becomes increasingly complex in the east. As shown in Figure 3, xenoliths of anorthosite are present within the ore. The anorthosite, which is located within the ore body or in the contact zones of the ore body, typically shows more alteration than the surrounding rock (Karlsen, 1997).

![Figure 3](image_url)  
**Figure 3** Outline of the mine site geology (taken from Karlsen, 1997), as highlighted in Figure 2.

Two major diabase dikes crosscut the ore body in a WNW-ESE direction. These have a straight appearance and mainly a vertical inclination. The largest one, the main dike (furthest to the south), is about 25 meters wide. As the main dike exits the ore body in the east, it forms a swarm of several smaller dikes extending from the main body. Two distinct faults, the Hommedal and the Tellnesvatn fault, and several smaller fracture systems cut the ilmenite ore. Studies, such as Karlsen (1997), show that there are some limited areas of heavy alteration that is related to the fracture and fault systems at the mine site, in both the ore and the anorthosite.
3 Geotechnical environment

3.1 Main lineaments

Based on aerial photos, the main lineaments have been identified and mapped in GIS to provide a clear understanding of the regional situation. These lineaments may also be part of several structural regions. Karlsen (1997) summarized the main regional lineaments present in the Tellnes district as the following:

- Åna-Sira (WSW-ENE)
- Jossingfjord (NE-SW)
- Hommedal (N-S)
- Crusher (NNW-SSE)
- Tellnesmyra (NW-SE)
- WNW-ESE

![Figure 4 Illustration of the main lineaments in the Tellnes district. (DEM image source: Google maps)](image)

From Figure 4, it is clear that there are lineaments that have a large influence on the local fracture systems in certain areas near the mine site: The WNW-ESE lineament causes persisting and small spaced joint set that runs almost parallel to the orientation of the open pit slope in both the hanging- and footwalls. It is also possible to distinguish (for example in the central portion of Figure 4) certain areas of the mine where the N-S trend tends to have smaller joint spacing. This may cause wedge failure if it is combined with another unfavourable joint set.

3.2 Detailed jointing

An assessment on the detailed jointing was carried out of the area of study shown in Figure 5. The area is enclosed between the two large lineaments that stretch in the WSW-ENE direction. In addition, the zone is limited to the hanging wall of the mine that is facing NE. Inside the selected area; it is possible to clearly identify 4 distinctive joint sets based on the inspection of high resolution aerial photos, as can be seen in Figure 5:
- The first joint set runs parallel to the slope face with orientation WNW-ESE, which is well correlated with one of the main lineaments (with the same name), and has distinctly identified spacing. The typical spacing identified through aerial photo is 10-20 meters.
- The second joint set runs in the NW-SE direction, with a typical spacing ranging from 25 to 50 meters. It is related to the Tellnesmyra and Crusher fracture system and shows a slightly larger spacing than joint set number one. This set is important in terms of influence on stability of the area, since events of sliding planes have occurred in the past.
- The third joint set identified runs in the N-S direction and it is related to the Hommedal fracture system. This joint set is widely spaced with a typical spacing ranging between 80-120 meters. It is important to note that this joint always intersects both the first and second joint systems, and is systematically distributed in the studied area.
- The last joint set is related to the Åna-Sira fracture system and, in this case, defines the north and south boundary of the area, running in the ENE-WSW direction. It is expected that this set is not very distinctive, since its typical spacing is around 400-500 meters and, as has been seen in the mine during fieldwork, there are not many joints parallel with the main lineaments in the considered area.

![Figure 5 Area of study for the structural model. It is clear to see that there are 4 major joint sets present in the area, with orientation N-S, WNW-ESE, NW-SE, and ENE-WSW](image)

4 Analysis of joint systems information from different sources

The main objective of this analysis is to provide a proof of the reliability when combining data from different sources in order to develop a structural model for the selected area of the pit shown in Figure 5. The following procedure represents only a test of the techniques, and it is not intended to provide a detailed analysis of joint condition, orientation and the structural domains in the pit.

Three main sources of structure data were available to analyse the joint systems in the area of interest:

- Remote sensing using photogrammetry supplied by Sirovision (software available at sirovision.dataminesoftware.com).
- Field mapping of different benches along the slope face and tunnels in the surrounding area.
• Orientation mapping of discontinuities obtained from acoustical televiewing log of four boreholes inside the area, done by RudenAS (3 were core drilled and 1 was hammer drilled).

Twelve 3D images were analysed using the software to provide information about the jointing condition. The results obtained were information about dip and dip direction of joints. Data were obtained by tracing the contours of fractures that can be identified in the 3D images. Based on the images, the following analysis steps were taken:

1. Geo-reference each of the positions for the 3D images analysed.
2. Generate a zone of 100 meters (based in the average maximum spacing of the previously identified joint systems) around this position in order to have information about which boreholes or field mapping data was in the neighbourhood of the 3D image.
3. Analyse, via pole-, rosette- and density plots, each of the 12 sets of information (remote sensing, field mapping and acoustical televiewing of boreholes) for the joint sets present in each data source separately, and check how they correlate with each other.
4. Combine all the data in each of the 12 sets related in order to have a unique rosette for each zone. Then distinguish the sets discussed above using the joint sets identified in the 3D images, from field mapping (this is the slope face and also the tunnels present in the area), and from four boreholes selected in the area (Figure 4 & 5).
5. Finally analyse the resulting pole, rosette and density plot defining each of the distinct joint sets and assess whether there exist correlation in terms of dip and dip direction with the regional lineaments (or following strike and dip).

<table>
<thead>
<tr>
<th>Sirovision</th>
<th>Boreholes</th>
<th>Field Mapping</th>
</tr>
</thead>
<tbody>
<tr>
<td><img src="image" alt="Sirovision Boreholes Field Mapping" /></td>
<td><img src="image" alt="Sirovision Boreholes Field Mapping" /></td>
<td><img src="image" alt="Sirovision Boreholes Field Mapping" /></td>
</tr>
</tbody>
</table>

Figure 6  Rosette plots for the discontinuities identified in the 3D image remote sensing in Sirovision (left), borehole information from acoustical televiewer (centre), and field mapping of the slope face and tunnels (right) in the interest area

The analysis of pole, contour and pole density plots were done in Dips (available at [www.rocscience.com](http://www.rocscience.com)). The number of available data (in terms of identified joints with its respective measurement of dip and dip direction) was 1059 points in Sirovision, 810 points from borehole imaging and 81 in field measurements (49 from geotechnical windows on the slope face and 32 joints identified during tunnel inspection). Figure 6 shows the resulting joint rosette plots from all the three sources of information. Average dip and dip direction of each of the joint sets were recorded. These findings will be explained in detail in chapter 5.

The joint rosette plotted based on field mapping data also identifies three main joint sets and all of them correlates quite well with the ones found in the Sirovision plot. J1 that goes in direction WNW-ESE, J2 oriented in direction NW-SE and slightly declining and merging into the direction of J1, and J3 going in the N-S trend, with a very small deviation to the NNW direction.
Finally, by looking in detail (and with the previous knowledge of the main lineaments) it is possible to identify some discontinuities aligned in the NE-SW direction both in the Sirovision and in the field mapping data. This joint set could be interpreted as J4, but the amount of data has not been considered enough to conclude that this represents the system of joint sets relevant to the pit slope. As the drillholes are orientated in the NE-SW direction and the slope face is NW-SE, NE-SW structures (i.e. the Åna-Sira formations) are uncommon in occurrence as well as probably underestimated in the mapping.

5 Analysis of fracture systems

Taking into account the joint sets recognized in the previous chapter and the direction of the main lineaments described in chapter 3, all of the identified joint sets have been found to have a consistent correlation with the regional trends shown in Figure 4. Summary of the statistical variating of the fracture systems are presented in Table 1.

The first joint set (J1) has an overall average Strike/Dip of N113E/81. In terms of dip direction it is possible to find joints with dip direction in both ways (i.e. close to 023° and 203°) and it is very well correlated with the WNW-ESE fracture system in terms of the apparent strike of the lineament. It is clear from Figure 5 that this joint set is the one with the smallest spacing, and thus it is considered the most important in terms of the stability of the pit slope. Since the dip angle is more similar to this set, the joint set identified through the analysis of boreholes is considered to belong to J1.

Table 1 Statistical distribution of Dip/Strike for the main joint sets found in the studied area of the Tellnes open pit mine

<table>
<thead>
<tr>
<th>Joint set</th>
<th>Sirovision</th>
<th>Boreholes</th>
<th>Field Mapping</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Dip (direction)</td>
<td>Strike</td>
<td>Dip (direction)</td>
</tr>
<tr>
<td>J1 avg</td>
<td>85 (NW/SE)</td>
<td>N109E</td>
<td>78 (NW/SE)</td>
</tr>
<tr>
<td>Max</td>
<td>90</td>
<td>137</td>
<td>90</td>
</tr>
<tr>
<td>Min</td>
<td>78</td>
<td>84</td>
<td>59</td>
</tr>
<tr>
<td>Stdv</td>
<td>3</td>
<td>14</td>
<td>8</td>
</tr>
<tr>
<td>J2 avg</td>
<td>46 (NE)</td>
<td>N133E</td>
<td>-</td>
</tr>
<tr>
<td>Max</td>
<td>57</td>
<td>153</td>
<td>-</td>
</tr>
<tr>
<td>Min</td>
<td>34</td>
<td>118</td>
<td>-</td>
</tr>
<tr>
<td>Stdv</td>
<td>6</td>
<td>10</td>
<td>-</td>
</tr>
<tr>
<td>J3 avg</td>
<td>78 (E/W)</td>
<td>N177E</td>
<td>-</td>
</tr>
<tr>
<td>Max</td>
<td>90</td>
<td>191</td>
<td>-</td>
</tr>
<tr>
<td>Min</td>
<td>60</td>
<td>161</td>
<td>-</td>
</tr>
<tr>
<td>Stdv</td>
<td>9</td>
<td>8</td>
<td>-</td>
</tr>
</tbody>
</table>

The second joint (J2) set has a global Strike/Dip orientation of N140E/49NE and, again, a good correlation with the Tellnesmyra and Crusher lineaments. The dip direction in this set is well defined, and it has an average dip of 49° in the NE direction. This is comparable with the description an unfavourable joint system in the hanging wall by Nilsen and Ballou (2006), who stated that typical strike of these joints is N115-160°E, and the dip is typically 40-50° NE. In addition, Botsialas and Mass (2014) describe a set of fractures with an approximate Dip/DipDir of 65/045 that belongs to the Crusher fracture system. This system is oriented oblique to the longest axis of the pit. It is clear that the main lineaments is related to the joint set, but in some cases, they could interact and lead to an intermediate joint set that is the product of the interaction between these two structural trends.

The third joint set (J3) has a general Strike/Dip orientation of N178E/79 but here again; it is possible to find joints with a dip direction in both ways (i.e. close to 268° and 88°). This third system of joints is clearly aligned in the same direction as the Hommedal fracture system.
As a final point, evidence of distinctive discontinuities in the direction of the Åna-Sira lineament have not been found as a joint set of importance in the area, but as it has been described in the previous chapters, there are some structures aligned in this direction (named as J4). It seems that there is no great influence of the trends of this system in the jointing of the zone, because they do not generate an associated joint system in the space between two mapped lines. In fact, this lineament does not influence the discontinuities in the rock mass between them, only creating local zones of weakness around the faults themselves.

6 RQD versus fracture frequency

Several attempts to correlate the Rock Quality Designation (RQD) with the amount of fractures per unit of surface or volume were made by some authors in the past. Palmstrom (2005) provided a correlation between various measurements and shapes of block size with the volumetric joint count. Priest and Hudson (1981) derived a correlation between the RQD and the number of fractures per meter (FFm). The correlation, which is defined by Eq.1, is the more well-known:

$$ RQD = 100e^{-0.1FFm(0.1FFm + 1)} $$

However, no extensive research has been done in addressing the quality of the correlation, taking in account that a certain number of joints could be distributed in many (or infinite) different positions along one cubic meter of rock block.

To try to solve this issue, a statistical simulation was done where random values for the position of a given set of fractures along a linear meter block were generated, with values ranging from 1 to 50 FFm. The simulation consisted of 50,000 iterations using VBA code. RQD values were calculated and recorded in each iteration in order to achieve distribution of values for any given number of fractures between 1 and 50.

It was found that the estimated RQD values for each number of fractures followed a normal distribution. Hence, for each value the average was calculated and plotted against its respective value of fractures per meter. The result gave a slightly more conservative correlation (Figure 7) than the one found by Priest and Hudson (1981). A brief run of the Anderson-Darling test of normality shows p-values ranging from 0.006 to 0.579, typically decreasing as the number of simulated fractures increases. It is clear that the generated data follows a normal distribution in lower values of fractures per meter (1 to 20), while it tends to be truly random in greater values (20 to 40, p-value around 0.1 and more than 40 with p-value less than 0.03).

When the RQD or the FFm is estimated based on the other one, it is very useful to be able to measure the confidence interval for the estimation. Therefore, the standard deviation of the calculated RQD values was plotted against the RQD value itself (Figure 7), so it is possible to quantify a confidence interval of 68% or 95%, as desired, for a given estimation. The maximum standard deviation (11.49) is obtained at 23 fractures per meter with an average RQD of 28.

Since it is possible to have a measure of the number of fractures per meter both from the Sirovision 3D images and the acoustical inspection of the boreholes, RQD (and its associated uncertainty) can be derived. This correlation aims to provide another source of estimation of the fracture density, other than the visual approximation/calculation from the field mapping, which could provide input into the calculation of the RMR or Q values.
7 Geometry of failure modes

The geometry of potential failure modes in the hanging wall of the studied area was analysed considering the dip and dip direction of the joint sets identified. The following chapter is only illustrative, and is meant to analyse only in a general way, potential failure modes in the defined zone. However, it is emphasized here that the result of this analysis does not ascertain that a failure will occur.

This analysis only focuses on the orientation and dip of the joint sets and the interaction with the slope, which has an overall Dip/DipDir of 55/045 and a bench face angle of Dip/DipDir 85/045. It is well documented that because of unfavourable combinations for the orientations of major joints and faults, local stability problems have occurred in several places in the upper pit walls. Most of these incidents have occurred during heavy rainstorms and during periods of repeated freezing/thawing (Nilsen and Ballou, 2006).

Referring to the first case (left) in Figure 8, J2 and the bench face angle for the slope are defining a typical geometry of planar failure. The average dip angle of $49^\circ$ to the NE of J2 and an orientation almost parallel to the slope face defines a plane that could daylight above the toe of certain benches. Nilsen and Ballou (2006) stated that, particularly in the hanging wall, there are very continuous joint sets in bench scale, with intermediate dip to the NE, which represents a risk of plane failure. These surfaces contain slippery chlorite schist that make it easy for the rock masses to slide once the supporting rock mass is further excavated (Karlsen, 1997). Figure 8 left shows an analysis of the risk of plane failure in bench scale (slope face at Dip/DipDir of 55/045). J2 would generate a plane that dips very close to the orientation of the overall slope itself. It is possible to see evidence of this in the upper benches of the area, where in the past a persistent joint generated plane sliding in many benches individually, but not in the entire slope.

Furthermore, J1 in combination with J3 and the bench face angle define the geometry of a possible wedge failure in the bench. As can be seen in Figure 8 right, the overall size of the wedge is not expected to be of importance, but due to the possible variations given the standard deviations shown in Table 1 for both joint sets, some major wedges could be formed in certain areas. These two joint sets are also the probable cause of areas with blocky geometry that can be observed in certain benches. This is due to their interaction with a local joint set that has a shallow dip, which forms a tetragonal geometry, which induces the fall of individual blocks.

It is also possible to find local toppling failure due to the variation of the dip direction of J1. As this joint set includes dip directions both to the NE and to the SW, the latter is probably responsible for the local toppling failures when found. As this fracture system has an average dip of $85^\circ$, but can vary between $78^\circ$
(to define one face of a wedge) and $90^\circ$. Toppling failure could also be induced when the strike of $J_1$ approaches that of $J_2$, as it is possible to see in the boreholes rosette joint orientation.

In order to have a more detailed interpretation of the failure mechanisms, it is suggested that it could be worthwhile to try to calculate the average dip and dip direction/strike of each joint set by weighting each discontinuity by its persistence and/or aperture. This will help in the identification of the most unfavourable joint set in terms of the stability of the slope, with the aim of minimizing the bias introduced by smaller discontinuities into the overall orientation.

## 8 Conclusions

The three different approaches in discontinuity mapping that were presented in this article gave good confidence level in defining the representative orientation of the joint systems. The analysis also demonstrated that there is a good correlation between the joint sets found and the main lineaments that was mapped using aerial photos. Focusing from the perspective of a bottom-up approach, it is also possible to identify not only the orientation, but also the average dip of the most important and influencing lineaments in an area of the open pit, based on the characteristics of the locally identified joint sets. Therefore, it is possible to conclude that the WNW-ESE fracture system is correlated with the $J_1$ set and has an overall average Strike/Dip of N113E/81. The Tellnesmyra and Crusher lineaments are related to the $J_2$ set, with an average orientation of N140E/49. Finally the Hommedal fracture system is associated to the third joint set ($J_3$), and it is interpreted to have an average orientation of N178E/79. This approach of making a structure model will be further improved by carrying out analysis in other selected areas of the mine and will be published in the future.

The correlation attempt made between RQD and FFm provided a confidence measurement when calculating one parameter from the other. The equation derived from the correlation is intended to be used in the future to characterize the rock mass with RQD values that can be derived from 3D images (using Sirovision) or the acoustical televiwer log of the boreholes.

Finally, a brief analysis of the main joint sets and their interaction with the slope face identified two possible failure modes in the bench scale: Set $J_2$ and the bench face angle describe a planar sliding failure while a combination of sets $J_1$ and $J_3$ form wedges with the bench face angle. It is also predicted that due to variations in the Dip and Dip Direction of all joint sets, local toppling may occur (when interacting with a
Building a structural model for assessing potential pit slope failure—a case of a Norwegian mine

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sub horizontal joint set) in certain areas on a bench scale. However, it is also stressed that any potential sliding is governed by not only the discontinuity and slope face orientation but also the overall characteristics of the discontinuity surface and hydrogeological conditions of the pit-slope. Therefore, more assessment of these aspects will be carried out in the days to come.

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References


Comparison between dynamic ground support methods (dynamic bolting)

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Abstract

The target of this research was to compare deformation of dynamic rock bolts to traditional cable bolts in challenging rock mass conditions in situ. The chosen test heading had variable zones with GSI classes from good to weak rock mass. Two types of dynamic bolts were chosen for testing; D-bolt (Normet International) and NMX Dynamic bolt (Nybergs Mekaniska Verkstad). Both bolt types yield maximum of 10% when under pressure caused by rock movements. Dynamic bolts were 3 m long and traditional cable bolts were 6 to 8 m long. The test heading was divided in 10 m long sections with different bolt types. All sections and bolt types had a same drill pattern. The test heading was monitored for a period of 12 months, during which it was 3D scanned three times; immediately after bolt installation, after 6 months and after 12 months. From the scans 17 cross sections were chosen for further research. Cross sections were compared to previous scans to examine the deformations. Deformations were measured and weighted with circumference to create a deformation figure (DF) for each cross section. Cross sections were compared by DF’s and D-bolt had the least deformation and cable bolt had the biggest deformation. However, the results were greatly influenced by cross section’s GSI class, and further research is necessary for comprehensive and comparable results.

1 Introduction

The Kemi mine is located in Keminmaa County, North East of the town of Kemi in an area known as Elijärvi. It is owned by Outokumpu Chrome Oy and it’s part of the Outokumpu chain of integrated ferrochrome and stainless steel manufacturing in the Kemi-Tornio region. The mine itself is a large underground operation with a production capacity of 2.7 million tons of ore per annum. It is also the European Union’s only chrome mine, and at present (2015) the mine extracts 2.0 million tons of chrome ore. The Chrome ore body is long and narrow sheet at vertical 70 degree angle (Figure 1).

Figure 1. Kemi Mine. Old open pits, underground mine and the ore body. (Outokumpu Chrome Oy).
The chrome ore is mainly mined above level 500, using a bottom up transvers benching method (Figure 2). From the stopes the ore is transported to ore shafts, crushed with underground primary crusher and hoisted up to surface. The empty stopes left behind are then filled with consolidated waste rock. In addition to the Elijärvi and Viia ore bodies, the Kemi mine is also developing the mining area, starting at level 550 mining upwards in Surmaoja area.

Challenging rock conditions, squeezing ground and faults are part of daily life at Kemi mine. Ground support has been developed to more dynamic and right time support methods, including meshing, expansion shell bolts, fiber shotcrete and partly non-grouted cable bolts. Ground support methods are continuously developed to ensure safe working conditions. Dynamic bolts can be an option to traditional cable bolts. This research aims towards this goal and uses two different types of 3 m long dynamic bolts to compare their behavior with 6-8 m long cable bolts by their performance in preventing tunnel transformations.

2 Materials and methods

In previous studies rock bolt anchorage performance of dynamic bolts have been evaluated by laboratory scale tests, (Chen 2014) or real life movements have been tried to implement theoretical models (Malmgren & Nordlund 2008). In this work, tunnel sections were supported by 3 different bolt types and wall deformation was measured.

Test bolt types

Cable (steel strand) bolt is standard 7 strand cable (diameter 15.2 mm and tensile load 265 kN) with 3.5 % yield. Drill hole diameter is 51 mm and last 1.5 m of cable bolt is not grouted. Estimated maximum elongation is close to 5 cm. Cable bolts length varied from 6 meters to 8 meters.

D-bolt (manufactured by Normet) is a smooth steel bar with a number of flattened anchor points along its length. (Figure 3) Smooth sections between the anchor points can yield when subjected to rock deformation. D-bolt of 22 mm diameter has a shear load of 210 kN, a tensile load of 250 kN and a yield load of 190 kN, according to the manufacture’s laboratory tests. (Normet International Ltd. 2014) Tested D-bolt has a diameter of 22 mm and length of 3 m.

NMX Dynamic bolt (manufactured by Nybergs Mekaniska Verkstad) has M24 thread, is made from rebar and covered partly by smooth steel pipe. (Figure 2) Bolt can yield inside of this steel pipe while rough rebar surface at both ends anchors bolt to the rock. NMX Dynamic has a shear strength of 235 kN, a tensile strength of 199 ± 8 kN and a yield strength of 170 ± 3 kN (Malmfalten AB 2014). Tested NXM Dynamic bolt has a diameter of about 22 mm and length of 3 m.
Both dynamic bolt types should yield maximum 10% with a total elongation up to 15 cm for a 3 meter bolt.

![D-bolt](image1.png)

![NMX Dynamic](image2.png)

Figure 2. Tested dynamic bolt types. Above in the picture is D-bolt, a smooth steel bar with a number of flattened anchor points, and below NMX-Dynamic, a rebar covered partly by smooth steel pipe. (H. Harju paraphrasing Normet International Ltd. 2014 and Malmfalten AB 2014).

**Test heading and test set up**

A production heading with a length of approximately 100 meters was chosen for testing from an area with active deformations. Rock mass quality varies from weak to good GSI-classification (Geological Strength Index) and at least two fault lines cross the test heading. During tunneling, heading was supported by meshing, Swellex-type bolts and shotcrete. Test heading was divided into 10 meter sections. 1st section was supported by cable bolts (steel strand), 2nd by NMX-dynamic bolts, 3rd by D-bolts, 4th by cable bolts, and so on. Each 10 meter section consisted installation of five bolt fans and each fan had 10-12 bolts depending on heading profile. This cycle was repeated with all the test bolt types for whole heading (Figure 3). Installation work was completed in January 2015.
Figure 3. GSI-classification of the test heading, test set up with analysed cross sections and used bolt fan (Outokumpu Chrome Oy).

**Analysing methods**

The rock deformations of the test heading were studied with 3D scanning. Total of three scans were performed in year 2015; 1st scan immediately after bolt installation in January, 2nd in June and 3rd in December.

Total of 17 cross section scans were picked for further analysis by their positions across the test heading. (Figure 3) The three scan results were compared on a light board and the deformations were marked and measured according to scale (Figure 5).
Figure 4. Example of a tunnel wall deformation. Deformation after 6 months is marked in the picture with purple colour. Deformation after 12 months is marked in the picture with red colour. The measured deformation thickness is marked with colour respectively.

To be able to compare the deformations between cross sections with different circumferences, a figure describing the deformations was developed (Equation 1). Deformation figure (DF) takes account of different circumferences and it’s only calculated for two walls and roof, so possible differences in tunnel floor are eliminated. DF was calculated for walls and roof and then the three DF’s were summed together to make a one figure to describe each cross section. The smaller the DF, the smaller the deformation.

\[
DF_i = \frac{h_d}{c_{cs}/4}
\]

in which
- \(DF_i\) = Deformation Figure,
- \(i\) = Section = left wall, right wall or roof,
- \(h_d\) = Thickness of Deformation,
- \(c_{cs}\) = Circumference of Cross Section

### 3 Results

According to scanning results, tunnel walls were deformed inwards at maximum 60 cm, during this one year period. Around 20 washer plates had also failed (nuts had gone through plates and sank into tunnel walls). Clearly several bolts have also broken inside the rock and at least one bolt’s broken part had dropped to the road (Figure 5). Unfortunately, total elongation and possible breakage cannot be verified from these bolts and only the most obvious failures are seen.
When comparing the average DF’s by bolt type, D-bolt gives the smallest average DF of 5.86 cm. NMX Dynamic gives average DF of 6.52 cm and cable bolt 7.26 cm. D-bolt gives the smallest deformation after test period of one year (Table 1). Results per analyzed cross section are also described in figure 7 below. In figure 7 deformation figures are considerable bigger in test heading areas with GSI-classes of weak and moderate compared to part with GSI-class of good.

Table 1. Tunnel wall deformation after one year and weighted deformation values for each cross section.

<table>
<thead>
<tr>
<th>Cross section</th>
<th>Bolt type</th>
<th>Deformation (cm)</th>
<th>Circumference (m)</th>
<th>Deformation figure (cm)</th>
<th>Total DF (cm)</th>
<th>Average DF per bolt type (cm)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Left wall</td>
<td>Roof</td>
<td>Right wall</td>
<td>Last scanning December 2015</td>
<td>Left wall</td>
</tr>
<tr>
<td>CS 2</td>
<td>NMX Dynamic</td>
<td>3</td>
<td>15</td>
<td>10</td>
<td>33.03</td>
<td>0.36</td>
</tr>
<tr>
<td>CS 3</td>
<td>NMX Dynamic</td>
<td>8</td>
<td>15</td>
<td>14</td>
<td>41.88</td>
<td>0.76</td>
</tr>
<tr>
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<td>NMX Dynamic</td>
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<td>18</td>
<td>12</td>
<td>21.86</td>
<td>0.91</td>
</tr>
<tr>
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<td>NMX Dynamic</td>
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<td>13</td>
<td>3</td>
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<td>12</td>
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<td>3.83</td>
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<td>20</td>
<td>15</td>
<td>40.25</td>
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<td>30</td>
<td>15</td>
<td>41.97</td>
<td>4.10</td>
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<tr>
<td>CS 10</td>
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<td>4</td>
<td>16</td>
<td>8</td>
<td>23.93</td>
<td>0.67</td>
</tr>
<tr>
<td>CS 11</td>
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<td>4</td>
<td>10</td>
<td>4</td>
<td>19.57</td>
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<tr>
<td>CS 16</td>
<td>D-bolt</td>
<td>12</td>
<td>10</td>
<td>6</td>
<td>19.73</td>
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<tr>
<td>CS 17</td>
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<td>15</td>
<td>20</td>
<td>10</td>
<td>23.18</td>
<td>2.59</td>
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<tr>
<td>CS 1</td>
<td>Cable</td>
<td>4</td>
<td>0</td>
<td>2</td>
<td>25.25</td>
<td>0.63</td>
</tr>
<tr>
<td>CS 6</td>
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<td>25</td>
<td>10</td>
<td>14</td>
<td>19.64</td>
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<tr>
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<td>15</td>
<td>6</td>
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<tr>
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</tr>
</tbody>
</table>
4 Discussion

In total 17 cross sections were compared, which shows on how different bolt types resists rock mass transformation.

Different rock mass qualities and GSI-classes of the test heading might influence the results (Figure 7). Cross section 1 is at stable area, where large blocks compressing downwards do not create extra pressure to tunnel walls. Cross sections 3-6 are at the weakest rock mass and the area also has two active faults. Area between cross sections 10 and 15 should be most uniform in the rock mass quality aspect (Figure 6). Further research is necessary to eliminate the effect of differences in rock mass quality and to get comparable results between different bolt types. This could be similar research performed in test heading with more uniform rock mass quality or development of figure describing the relativity of DF and GSI-class.

5 Conclusion

Results show that, after one year after installation, the D-bolt had the smallest average deformation figure per bolt type and the traditional cable bolt had the biggest average DF per bolt type. GSI-classes of the test heading are varying from good to weak and this might have some effect on the results.

However, the result indicates that even doubling the length of cable bolts (when compared to other two bolt types) did not increase the support effect. In addition, it did not helped to slow down large scale movements or tunnel wall compression.

References


Critical embedment length of fully grouted rebar bolts

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Abstract
The maximum reinforcement depth of a fully grouted rockbolt theoretically is equal to the length of the bolt minus its critical embedment length. The critical embedment length is defined as the longest encapsulated length of the bolt with which the bolt can be pulled out of the grout without failure of the bolt shank. A series of laboratory tests were carried out to find the critical embedment length of a type of cement-grouted rebar bolt in three different water-cement (w/c) ratios: 0.40, 0.46 and 0.5. It was determined that the critical embedment length of the bolt was approximately 25 cm for the w/c ratio 0.40 (grout UCS 37 MPa), 32 cm for the ratio 0.46 (UCS 32 MPa) and 36 cm for the ratio 0.50 (UCS 28 MPa). The stress measurements on a bolt, with an embedment length that was shorter than the critical length, showed that the shear stress on the bolt exponentially decreases with the distance to the loading point before the pull load reaches the yield load of the bolt shank. The bond strength may be fully mobilized when the peak load is reached. The shear stress during slippage of the bolt shank, called the residual shear strength, is approximately uniform along the embedment length and reduces gradually with the increase of slippage displacement.

1 Introduction
Fully grouted rebar rock bolts are widely used for rock reinforcement in civil excavation and mining applications. The critical embedment length of this type of rock bolt, defined as the maximum embedment length prior to failure of the bolt shank under a pull load, is a concern for rock bolting design. For instance, the required anchoring length of a rebar bolt in the stable stratum is dependent on the critical embedment length of the bolt when a loosened rock block is stabilized with fully grouted rebar bolts. A series of laboratory pull tests were recently carried out in accordance with the procedure described in the suggest method (Franklin et al. 1974), aiming to determine the critical embedment length of rebar bolts grouted with cement mortars with different water-cement ratios. The results of the tests on the critical embedment length and the bond strength are reported in this article.

2 Specimens and testing arrangement
2.1 Rock Bolts and cement grout
The type of rebar bolt, 20 mm in diameter, is widely used in road tunnels in Norway (Figure 1). It is made of steel B500NC. The surface of the bolts is treated by hot zinc-galvanisation with a minimum thickness of 65 um and a coating of epoxy powder with a minimum thickness of 60 um for corrosion resistance. The characteristic yield and ultimate tensile loads of the bolt are 157 kN and 188 kN, respectively.

The grouting agent used in the tests is called Rescon zinc rock bolt cement mix. The cement mix is made of two components: cement (c) and silica (s). The weight ratio of the mix (cement and silica) to the cement in the mix is (c+s)/c = 1.7. The ratio of the water (w) to the mix (c+s) has the following relationship with the water-cement ratio (w/c):

\[
\frac{w}{c+s} = \frac{c}{c+s} \cdot \frac{w}{c} = \frac{1}{1.7} \frac{w}{c}
\]
2.2 Testing

The embedment length of the rock bolts was varied for different water-cement ratios. Three rock bolt specimens were pulled for every embedment length. Three cubic grout specimens, $100 \times 100 \times 100$ mm in size, were prepared at the time when the rock bolt was grouted and then tested on the same day of the rock bolt pull test to measure the uniaxial compressive strength (UCS) of the cubes.

Boreholes were percussively drilled with a 48 mm drill bit in a cubic concrete block with a dimension of $950 \times 950 \times 950$ mm. The UCS of the concrete is approximately 110 MPa. The ready-mixed cement grout was pumped into the hole and the bolt was inserted to a pre-defined depth. The curing time was scheduled to be seven days, but some of the rock bolts were pulled out after eight days. When a rock bolt was tested, a cylindrical spacer, collaring the rock bolt, was placed on the top of the concrete block, a hydraulic cylinder (jack) was placed on the top of the spacer, and finally a rock bolt plate and a barrel-and-wedge unit were placed on the top of the setup string to fasten the rock bolt (Figure 2). The distance from the surface of the concrete block to the barrel-and-wedge, that is, the freely stretched length of the rock bolt is 60 cm. The movement of the bolt head was measured with respect to the base of the hydraulic cylinder with an extensometer.
3 Test results and analysis

3.1 Test results

3.1.1 Water-cement ratio 0.5

The pull load-displacement curves of the representative rock bolts for four different embedment lengths are shown in Figure 3 for the water-cement ratio 0.5. All the bolts with an embedment length less than 400 m slipped in the grout after the peak load was reached. The yield load (170 kN) was reached for the bolts of 300 mm and 400 mm in embedment length. The bolt of 300 mm slipped in the grout after a short period of yield / hardening, but the bolt of 400 mm elongated in the steel to approximately 70 mm when the test was stopped. It was assessed that the bolt would soon fail in the bolt shank. The peak loads of the other two bolts with shorter embedment lengths were lower than the yield load of the bolt shank.

![Figure 3 Pull load–displacement curves of the bolts grouted with water-cement ratio 0.5](image_url)

3.1.2 Water-cement ratio 0.46

The pull load-displacement curves of the representative rock bolts for four different embedment lengths are shown in Figure 4 for the water-cement ratio 0.46. The bolts of 250 and 300 mm in embedment length slipped after the bolt shank yielded and when it was in the period of yield / hardening. The other two bolts with shorter embedment lengths slipped when their peak loads were lower than the yield load of the bolt shank.

3.1.3 Water-cement ratio 0.4

The pull load-displacement curves of the representative rock bolts for four different embedment lengths are shown in Figure 5 for the water-cement ratio 0.4. All the bolts with an embedment length less than 300 m slipped in the grout after the peak load was reached. Both the bolts of 200 and 300 mm in embedment length yielded in the shank. The bolt of 300 mm failed in the shank at the end of the test, while the shank of the bolt of 200 mm slipped after a short period of yield / hardening. The other two bolts with shorter embedment lengths slipped when their peak loads were lower than the yield load of the bolt shank.
3.1.4 UCS results of cement mortar

Three cubic specimens of the cement grout for every water-cement ratio were tested for UCS after seven-day curing. The size of the specimens is $100 \times 100 \times 100$ mm. The UCS results of the cubic specimens are presented in Figure 6 for the for the three water-cement ratios. The UCS decreases linearly with the increase in water-cement ratio. The regression equation is as follows:

$$UCS = 77 - 99 \times (w/c)$$  \hspace{1cm} (2)
3.2 Analysis

The ultimate pull loads of the bolt specimens are plotted against the embedment lengths for the three water-cement ratios in Figure 7. The points representing slippage fit a regression curve and the abscissa of the intersection of the regression curve with the ultimate load of the bolt shank, 200 kN, is defined as the critical embedment length of the bolt for the given water-cement ratio. The critical embedment length of the bolt is determined to be 360 mm for the water-cement ratio 0.5 (Figure 7a), 320 mm for the water-cement ratio 0.46 (Figure 7b) and 250 mm for the water-cement ratio 0.4 (Figure 7c). The critical embedment length of the bolt, $L_c$ in mm, is approximately linearly related to the water-cement ratio as well as to the UCS of the cement mortar as follows (Figure 8):

$$L_c = 110 \times (w/c) - 19.1$$

$$L_c = 72.5 - 1.28 \times UCS$$

Notice that the critical embedment lengths presented in Figure 8 were obtained under strictly controlled laboratory conditions, which guaranteed satisfactory grouting qualities. The critical embedment length in the field could be longer than the laboratory-obtained value, taking into account variations in grouting quality as well as rock mass quality. Therefore, a safety factor between two and four should be used for the critical anchoring length of rock bolts in the support design (Littlejohn 1992).
Figure 7  Ultimate pull load versus the embedment length of the bolt for water-cement ratio: (a) 0.5, (b) 0.46 and (c) 0.4
4 Stress distribution along embedment length

The axial stresses along the embedment length were measured through strain gauges attached to a bolt specimen that was grouted in a cement mortar with a water-cement ratio of 0.6. The embedment length of the bolt specimen was 300 mm. The bolt shank yielded during the pull test, but eventually slid in the grout at the end of the test when the shank was subjected to hardening, see Figure 9. Points A1-A2 on the curve are in the stage of elastic deformation of the bolt, B marks the yield point, C the onset of the slippage of the bolt shank and D1-D2 in the stage of slippage. The axial pull loads along the embedded portions of the bolt are presented in Figure 10 for the pull test loading points marked on the curve in Figure 9. The average shear on the bolt surface can be calculated according to the formulas below:

\[
\tau = \frac{1}{\pi d} \frac{dP}{dx}
\]

(5)

where \(dP\) is the differential axial load, \(dx\) the differential bolt length and \(d\) the bolt diameter. The average shear stresses on the bolt along the embedment length are calculated from the data in Figure 10 using Eq. (5) and presented in Figure 11. The applied pull loads for curves marked by A1 and A2 are 39 and 80 kN, respectively, which are lower than the yield load (160 kN) of the bolt shank. The shear stress on the bolt decreases with the distance from the borehole collar where the pull load is applied (see Fig. 11a), indicating that no bond failure occurred and both the bolt shank and the bond responded to the applied load elastically.

The bolt shank yielded at B (161 kN). The shear stress on the bolt remains the highest (approximately 11 MPa) in the first 50 mm of the bolt section close to the borehole collar, but it becomes almost constant at approximately 9 MPa along the rest of the embedment length. This implies that the bond strength of the bolt-grout interface has been mobilized along the entire embedment length.
With the increase in the elongation of the bolt shank and steel hardening, the shear stress on the bolt section close to the borehole collar reduces and the bond strength becomes fully mobilized, as shown by the curve marked by C in Figure 11b. Slippage of the bolt shank started at C. In the period of slippage, the shear stress remains approximately constant along the embedment length, as shown by the curves D1 and D2 in Figure 11b, but the magnitude becomes smaller than that at the onset of slippage, marked by C. The magnitude of the shear stress reduces with displacement owing to accumulated bond failure.

![Figure 9 Pull load – displacement curve of a rebar bolt fully grouted with a cement-water ratio of 0.6](image_url)

![Figure 10 Axial pull load along the embedment length of the bolt for each applied pull test load](image_url)
Figure 11  Shear stresses on the bolt along the embedment length, (a) before slippage and (b) after slippage

5  Conclusion

A series of pull tests were conducted to determine the critical embedment length of a specific type of 20 mm rebar bolt that is widely used for rock support in tunnels and underground caverns in Norway. It was found that the critical embedment length is linearly proportional to the water-cement ratio and the UCS of the grout. The critical embedment length is approximately 25 cm for grout of UCS 37 MPa (water-cement ratio of 0.40), 32 cm for grout of UCS 32 MPa (water-cement ratio of 0.46) and 36 cm for grout of UCS 28 MPa (water-cement ratio of 0.50), for the Rescon zinc rock bolt cement used in the tests.

For an embedment length shorter than the critical length, the shear stress on the bolt exponentially decreases with the distance to the loading point before the pull load reaches the yield load of the bolt shank. The bond strength may be fully mobilized when the peak load is reached. The shear stress during slippage of the bolt shank, or call the residual shear strength, is approximately uniform along the embedment length and it reduces gradually with the increase of slippage displacement.

Acknowledgement

The authors would like to thank Mr Gunnar Vistnes and Mr Torkjell Breivik for their valuable assistance in the laboratory. The support by the Norwegian Road Authority–Statens vegvesen is gratefully acknowledged.

References


Discrete element modelling of steel wire mesh and rockbolt plate

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Abstract

The discrete element software YADE is used to simulate the load-displacement behaviour of high tensile strength steel wire mesh. A basic wire model consists of sphere and cylinder elements connected to each other to match the geometry of the wire mesh. Simple models were used to calibrate the micro-mechanical parameters used to represent the wire in the mesh. Models of bent wire were used to form a mesh. These models demonstrated the ability of the YADE models to capture in a realistic manner complex load-displacement behaviours of steel mesh. The development of discrete element mesh models is an important step in being able to simulate the role of mesh within a rock support system.

1 Introduction

Flexible support systems such as rockfall fences, mesh drapery, and bolted wire mesh are widely used for rock slope protection. Different types of wire mesh have been used in these systems. Chain-link mesh was originally the most commonly used type of mesh in North America. In the 1980s, double-twisted hexagonal wire mesh began to be widely used, due to its higher strength and resistance to unravelling when a wire breaks (Muhunthan et al. 2005). More recently, high-tensile strength steel rhombohedral wire mesh has been introduced to stabilize rock slopes (Roduner et al. 2010).

The use of bolted or pinned wire mesh has gained increased acceptance as a viable option for protecting slopes from excessive deterioration and ravelling due to weathering and for stabilizing steep soil and rock slopes. Bolted mesh systems consist of steel wire mesh, bolts/anchors, and steel plates. These systems are designed to lock unstable rock into its original position by using a pattern of bolts/anchors to pin the steel mesh to the slope surface. Numerous methods have been used to design these systems and controversy exists concerning what methods are applicable for these relatively flexible support systems (Blanco-Fernandez 2011). Some attempts have been made to conduct physical experiments and to apply numerical models to improve our understanding of the support mechanisms (Castro-Fresno 2008).

Finite element models have been applied to study the mechanical behaviour of wire mesh under different loading conditions. Sasiharan et al. (2006) tested the effect of individual elements, such as friction between mesh and rock and different locations of ropes on the stability of wire mesh systems. Roth and Ranta-Korpi (2007) studied the dynamic response of wire mesh under rockburst loading conditions. Castro-Fresno et al. (2008) estimated the resistance capability of wire mesh systems for slope stabilization and obtained the stress distribution in the wire mesh. Buzzi et al. (2015) studied the effect of block size and mesh geometry on mesh performance. Escallon et al. (2015) developed a model to simulate the loose connection between wires in chain-link wire mesh. However, the interaction of wire mesh and rock is a highly discontinuous problem because rocks can slide beneath the wire mesh and change the relative position of the wire mesh and the rock. Sometimes large deformations may occur. Therefore, the use of a discrete element modelling approach is more suitable than finite element models for these kinds of problems.

Bertrand et al. (2008) used the discrete element software, PFC3D (Itasca 2016), to model the behaviour of double-twisted hexagonal wire mesh. Each hexagonal section of mesh was represented by six particles (balls). The tensile interactions between the particles represent the forces in individual wires. The numerical model was calibrated using data from tensile strength tests on single wires and on sections of...
double-twisted mesh. Thoeni et al. (2013) improved the modelling of wire mesh by adding a more complex stochastic interaction between particles that represent the wires in the mesh. They used the discrete element software, YADE (Šmilauer et al. 2015), for their models. Dynamic tests show that this kind of model can simulate the behaviour of wire mesh and was successfully used to model a drapery system for rockfall protection (Thoeni et al. 2014). In these models, individual sections of steel wire are represented by two particles at each end with a numerical spring connection between the particles. The simulation of wires in a mesh was further improved by introduction of a discrete cylinder element. Based on the spar element proposed by Chareyre and Villard (2005), a cylinder element was developed by Bourrier et al. (2013) and extended by Effendzourou et al. (2016) to model deformable structures such as roots, grids, and membranes. Cylinder elements provide beam-like behaviour, such as tension, bending and twisting, and can also interact with other objects.

In this paper, the open-source discrete element software YADE is used to model the behaviour of rhomboidal steel mesh with cylinder elements. The numerical models for individual wires and sections of wire mesh consisting of bent and overlapping wires were created and calibrated using available laboratory test data. Considerable effort was devoted to simulation of bends in the wires and how one wire interacts with another wire at each bend. This work is the first stage in on-going research to model more realistically the interactions between wire mesh and rockbolt plates and between the mesh and deforming rock.

2 High tensile strength wire mesh

The modelling in this paper applies to GeoBrugg high-tensile strength steel wire mesh TECCO G65/3 (Cala et al. 2012). The ultimate tensile strength of a single wire is about 1.77 GPa and the steel has an approximate Young’s modulus of 200 GPa. The 3 mm diameter wires in the mesh create a pattern of rhombohedral 83 mm x 143 mm openings (Figure 1). The weight of the mesh is 1.65 kg/m². As pointed out by (Escallon et al. 2015), one single wire in the mesh is bent into a zig-zag pattern. Each “zig” (a bend in one direction) hooks with the wire immediately above it, and each “zag” (a bend in the opposite direction) with the wire below it, forming the rhombohedral pattern. The wires in the mesh have a loose contact at each bend.

Figure 1  TECCO G65/3 wire mesh
3 YADE discrete element modelling

The Discrete Element Method (DEM) simulates deformable objects using an assembly of discrete elements (Cundall and Strack 1979). These elements can interact with each other via contact forces that are generated by their relative motions and the assigned contact relationship. The motion of the elements is controlled by Newton’s second law. This method was implemented in the open-source software YADE. In this paper, the steel wires in a mesh were modelled using the sphere elements and the newly introduced cylinder elements, which are used to connect sphere elements.

The constitutive model of two spheres is the basis of other elements (Figure 2). The contact force is generated by the relative displacement (or overlap) or the relative rotation of the two spheres. The normal contact force $F_n$ and incremental shear force $dF_s$ are defined as:

$$F_n = k_n u_n$$  \hspace{1cm} (1)

$$dF_s = k_s u_s dt$$  \hspace{1cm} (2)

$$k_n = \frac{2E_1 R_1 E_2 R_2}{E_1 R_1 + E_2 R_2}$$  \hspace{1cm} (3)

$$k_s = \alpha k_n$$  \hspace{1cm} (4)

where:

- $k_n$ and $k_s$ are the contact stiffness associated with normal force and shear force respectively,
- $u_n$ is the relative displacement (or overlap) between two spheres,
- $u_s$ is the relative shear displacement,
- $dt$ is the time step,
- $E_1$ and $E_2$ are the Young’s modulus of the two spheres,
- $R_1$ and $R_2$ are the radius of the two spheres.

![Figure 2 Sphere-sphere interaction](image)

The bending moment $M_b$ and twisting moment $M_t$ are defined as:

$$M_b = k_b \Omega_{12}^b$$  \hspace{1cm} (5)

$$M_t = k_t \Omega_{12}^t$$  \hspace{1cm} (6)
where:

\( k_b \) and \( k_t \) are the contact stiffness with bending moment and twisting moment respectively,

\( \Omega_{12b} \) and \( \Omega_{12t} \) are the bending and twisting components of relative rotation.

The elastic limits are defined by:

\[
F_n \leq \sigma_n^l A
\]

\[
F_s \leq F_n \tan \varphi + \sigma_s^l A
\]  
(7)

\[
M_b \leq \frac{\sigma_n^l I_b}{R}
\]  
(8)

\[
M_t \leq \frac{\sigma_s^l I_t}{R}
\]  
(9)

where:

\( \sigma_n^l \) and \( \sigma_s^l \) are the tensile and shear strengths,

\( I_b \) and \( I_t \) are the reference polar and bending moments of inertia,

\( A \) is the reference surface area,

\( R \) is the minimum radius of the two spheres.

According to Effeindzourou et al. (2016), a single cylinder behaves like a classical discrete element. The cylinder elements can interact with other objects and with other cylinders. The cylinder can deform in the axial direction, but it cannot bend. A cylinder element corresponds geometrically to the Minkowsky sum of a polyline and a sphere (Figure 3). Two or more cylinders can be connected at one particle (sphere), which is often called a node. A cylinder can rotate and twist at the node, thus by using multiple cylinders connected by nodes, a model of steel wire can be created that allows the wire to both elongate and bend.

The axial deformation of the cylinder is defined by the positions of the two nodes at each end. Two or more connected cylinder elements behave like a beam, whose constitutive behaviour contains tensile and shear forces, as well as bending and twisting moments. The node connecting two cylinders acts like a virtual rotational spring. For a single cylinder element, the contact stiffnesses are defined as:

\[
k_n = \frac{E_s A}{L}
\]  
(11)

\[
k_t = \frac{G I_t}{L}
\]  
(12)

\[
k_s = \frac{12E_s I_b}{L^3}
\]  
(13)

\[
k_b = \frac{E_b I_b}{L}
\]  
(14)
where:

- $E$ is the tensile or compressive modulus,
- $E_b$ is the bending modulus,
- $G_t$ is the shear modulus associated with a twisting moment,
- $L$ is the distance between the centre of two spheres.

The formulation allows the model to capture sphere-sphere interaction, sphere-cylinder interaction, and cylinder-cylinder interaction (Figure 4). The sphere-sphere interaction uses the relative displacement or overlap and relative rotation of the two spheres to calculate the contact force and contact moment at the contact. In the sphere-cylinder interaction model, a virtual sphere within the cylinder is generated at the contact point (Figure 4a). The virtual sphere has the same radius as the cylinder and the position of the virtual sphere is at the projection of the contact point between the sphere and the cylinder on the segment connecting the cylinder nodes. The sphere-cylinder interaction turns into the more classical sphere-sphere interaction. In the cylinder-cylinder interaction model, two virtual spheres for each cylinder are generated at the contact point (Figure 4b). Similarly, the cylinder-cylinder interaction can be turned into a sphere-sphere interaction. In YADE, the material properties assigned to the spheres in cylinder elements control the behaviour of the cylinder while the material properties assigned to the cylinder control the interaction between unconnected cylinder elements.

The discrete element models of steel mesh were generated in YADE. A length of the wire is represented by a cylinder element connecting two nodes (spheres). The properties assigned to the sphere and the cylinder elements are listed in Table 1. A sphere is a cohesive frictional material while a cylinder is the frictional material because there is no cohesion between wires. The assumed Young’s modulus of the steel wire was
used as a model input. The Young’s modulus is converted to normal stiffness within YADE. For the sphere elements in the wire model, the shear stiffness was set to zero, as suggested by Bourrier et al. (2013).

Table 1 Parameters of sphere and cylinder elements use to represent steel wire

<table>
<thead>
<tr>
<th>Property</th>
<th>Symbol (units)</th>
<th>Spheres</th>
<th>Cylinders</th>
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<tr>
<td>Young's modulus</td>
<td>$E_c$ (Pa)</td>
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<td>200e9</td>
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<td>$k_s/k_n$</td>
<td>$\alpha$</td>
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<td>0.217</td>
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<tr>
<td>Density</td>
<td>$\rho$ (kg/m$^3$)</td>
<td>7850</td>
<td>7850</td>
</tr>
<tr>
<td>Friction angle</td>
<td>$\phi_c$ (°)</td>
<td>25</td>
<td>25</td>
</tr>
<tr>
<td>Tensile strength</td>
<td>$\sigma_t$ (Pa)</td>
<td>5.74e9</td>
<td>na</td>
</tr>
<tr>
<td>Shear strength</td>
<td>$\sigma_s$ (Pa)</td>
<td>5.74e9</td>
<td>na</td>
</tr>
</tbody>
</table>

4.1 Single wire loaded in tension

Published tensile test results (Cala et al. 2012) for a 100 mm length of wire with diameter of 3 mm gave a failure load of 12.9 kN with a maximum elongation of about 2% to 2.5%. This test was modelled with a single cylinder element connecting two spheres. The bottom end of the steel wire model was fixed and a vertical velocity was applied to the upper end of the wire. Figure 5 shows the force-displacement curve for the model. The steel wire modelled by the cylinder element shows a linear elastic relationship. The connection between the cylinder and one of the spheres broke at load of 12.9 kN, consistent with the ultimate tensile strength of the wire. The stiffness of the wire was 14 kN/mm, which corresponds a Young’s modulus of the wire of 200 GPa. The maximum elongation of the wire was 0.9 mm or a strain of 0.9%, which is less than the maximum elongation of the actual wire at rupture. This is to be expected since the wire model only captures the elastic response of the wire and in its current configuration does not simulate plastic yield of the steel immediately preceding rupture.

![Figure 5](image_url)
4.2 Single wire loaded in bending

A model of a wire consisting of five cylinders, each 20 mm long, connected to six nodes was constructed to test the bending behaviour of the wire model. The model represents a 100 mm length of 3 mm diameter wire. One end of the horizontal model was fixed, while a vertical load, \( P \), was applied to the other end. Figure 6 shows the bending curve of the wire under forces of 30, 50, and 70 N. Also plotted are the analytical shapes of the deformed wire assuming it behaves as an elastic cylindrical beam. The figure shows that the wire model can deform in a similar manner to an elastic beam although the modelled deformations are slightly more than the analytical solution. The difference is that the wire is assumed to be homogenous and isotropic in the analytical model, while the numerical model used only five connected cylinders to represent the wire.

![Bending test of a wire model made from five cylinders](image)

Figure 6 Bending test of a wire model made from five cylinders

The discrete element formulation in YADE can allow for simulation of dynamic or quasi-static behaviours. To illustrate a dynamic response, the time-dependent position of the loading point in the bending wire model is plotted in Figure 7. The damping used in the model is 0.2. The instantaneous application of the load on the end of the wire causes the wire to vibrate down and up and because the model includes a damping coefficient the magnitude of the vibrations gradually decrease until a stable deformed shape of the model is achieved. Then after the load is instantaneously removed, the wire vibrates again and eventually returns to the initial geometry. This model shows no permanent deformation because the wire responds in an elastic manner.
Figure 7  Dynamic response of wire that is rapidly loaded and then unloaded (P = 70 N)

4.3 Numerical models of wire mesh

Using the basic discrete elements of spheres and cylinders, a model of wire mesh can be constructed by connecting the spheres and cylinders to match the geometry of the mesh. Multiple connected spheres and cylinders are used to capture the bend geometry where one wire in a mesh is bent around another wire. In 3D, the wire has both a bend and a slight twist at this section of the mesh. Figure 8 shows a close-up view of the YADE model of two bent wires within a mesh. The bend in each wire is represented by four spheres and 3 cylinders.

Figure 8  Model of two wires bent around each other

The generation of the wire mesh is based on the following steps. First, one row of wire is generated by following the zig-zag pattern applicable to the geometry of the specific mesh type. Second, a second row of wire is created, which hooks with the first row to form the rhomboidal pattern. This step can be repeated to create a large area of wire mesh.

A series of numerical simulations were performed to measure the capability of YADE discrete element models to capture key load-deformation behaviours of wire mesh. A model of a small section of mesh was loaded in different directions and with different constraints on the boundaries of the mesh. Two models were loaded in the longitudinal (or stiff) direction and a third model was loaded in the transverse direction.
4.3.1 Loading in longitudinal direction with edges of mesh laterally constrained

A numerical model was configured such that the mesh was loaded in the longitudinal direction (vertically) at the top with the bottom of the mesh fixed into position. The loads were applied to the upper-most spheres in each of the two top wire bends. The bottom of the mesh was fixed by fixing the lower-most spheres in each of the two bottom wire bends. Both lateral edges of the mesh were constrained such that they could only move vertically. This was accomplished by restricting the motion of spheres in each of the two wires on both edges of the mesh to vertical motion. The vertical force versus vertical displacement for this model is shown in Figure 9. The inset in this figure shows the geometry of the mesh at a specific stage of the loading. The boundary conditions on the model combined with the geometric configuration of the two wires create a stiff load-displacement response and the mesh retains the rhombohedral pattern and only deforms a small amount before a high tensile force is developed in the wires and a wire breaks. The peak tensile force in the mesh is 40 kN and this occur at a displacement of 4.2 mm. This load capacity arises from the tensile strength of four wires inclined with respect to the longitudinal loading direction. If the forces are assumed to be carried by a simpler 2D geometry matching the wire orientations in the mesh, the predicted peak load would be:

$$F_{\text{mesh}} = 4 \times 12.9 \times \cos\left(\frac{49^\circ}{2}\right) = 47kN$$

This value is higher the predicted load of 40 kN because the mesh wires carry higher loads near the bends, i.e., the wires break at the bends.

![Figure 9](image.png)

**Figure 9** Vertical loading with sides only allowed to slide vertically

4.3.2 Loading in longitudinal direction with edges of mesh unconstrained

The numerical model shown in Section 4.3.1 was modified such that both lateral edges of the mesh were unconstrained and thus were free to move in any direction. The load applied to the top of the mesh was in the longitudinal or vertical direction only and the loading points were free to move laterally as the mesh deformed (as they were in the model in Section 4.3.1). Freeing the lateral boundaries of the mesh permits much more deformation. The vertical force versus vertical displacement for this model is shown in Figure
10. The inset in this figure shows the geometry of the mesh at the beginning of the test and at a later stage of the loading when the deformation reached 8.5 mm. The much higher longitudinal displacement occurs because the mesh can deform in the transverse direction.

Figure 10  Vertical loading with sides free

In this test, the both lateral edges were free and both lateral ends of the blue wires were allowed to slowly slip out from the hook during the test. There were only two wires working after that. So at failure, the load carried by the mesh is assumed to be twice the load capacity of a single wire 25.8kN. This value is also higher than the numerical results of 23.5 kN because the mesh wires carry higher loads near the bends, i.e., the wires break at the bends.

4.3.3  Loading in transverse direction with top and bottom of mesh unconstrained

The numerical model presented earlier was modified again such that loading was applied in the transverse or horizontal direction. The wires at one side of the mesh were fixed while the loading was applied to the opposite side. The other edges of the mesh were free to deform in any direction. The horizontal force versus horizontal displacement for this model is shown in Figure 11. The inset in this figure shows the geometry of the mesh at the beginning of the test and at a later stage when the elongation reached 150 mm. With this model configuration, the wires could simply straighten out at the bends allowing for very large displacements at small loads. After the wires deformed enough to become aligned nearly parallel with each other and the loading direction, the load carried by the wires rapidly increased until the point of failure. At failure, the load carried by the mesh was approximately twice the load capacity of single wire.
4.3.4 Effect of friction angle on the mesh load-displacement response

The wire mesh model shown in Section 4.3.1 was used to examine the sensitivity of the model response to the assigned values of the friction angle to the different element types. Figure 12 shows the results when the friction angle assigned to the spheres is varied. The resulting mesh response shows that the load-displacement curve is either extended or truncated depending on the value used. Higher friction angles result in reduced peak displacements and forces. This is because the forces acting on the spheres of the bending cylinders increased as the friction angle increased and failure occurs at the bending cylinders. Figure 13 shows the results when the friction angle assigned to the cylinders is varied. For this model, the response is not very sensitive to the cylinder friction angle because sliding at the hook in the wire during the tests was very small.
4.3.5 Load-displacement response for mesh loaded by a rockbolt plate

A wire mesh model loaded by a rockbolt plate was created to examine the force-displacement response of the plate and the deformed shape and failure mechanism of the mesh. The model is shown in Figure 14. An array of connected sphere elements was used to simulate the plate. The parameters of these spheres are shown in Table 2. As these spheres are clumped together, the plate in this model is rigid. The mesh was fixed around its outer perimeter. The plate was centred 20 mm below the mesh and loaded in a direction perpendicular to the mesh. Gravity is not applied in the model.
Table 2  Parameters of sphere elements use to represent rockbolt plate

<table>
<thead>
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<th>Property</th>
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<td>Radius</td>
<td>$r$ (mm)</td>
<td>5</td>
</tr>
<tr>
<td>Young’s modulus</td>
<td>$E_c$ (Pa)</td>
<td>200e9</td>
</tr>
<tr>
<td>$k_s/k_n$</td>
<td>$\alpha$</td>
<td>0.3</td>
</tr>
<tr>
<td>Density</td>
<td>$\rho$ (kg/m$^3$)</td>
<td>7850</td>
</tr>
<tr>
<td>Friction angle</td>
<td>$\varphi_c$ ($^\circ$)</td>
<td>25</td>
</tr>
</tbody>
</table>

The force versus the displacement for the rockbolt plate is shown in Figure 15. For the first 60 mm of plate movement there is essentially no resistance by the mesh. After a plate displacement of 60 mm the force gradually increases. When the plate displacement reached approximately 260 mm, some steel wires near the rockbolt plate broke, resulting in a sudden force decrease from 75 kN to 18 kN. The upper inset figure shows the deformation of the mesh and broken steel wires at this stage of loading. The force on the plate recovers somewhat with further displacement as the loads are transferred to other wires in the mesh. When the plate displacement reaches about 288 mm, the force decreases dramatically again. As can be seen from the lower inset figure, more wires are broken resulting substantial damage to the mesh.

![Figure 15 Load-displacement response of the plate and deformation/failure pattern of the mesh](image)

4 Conclusion

Discrete element models of steel wire mesh were created using cylinder elements connected to sphere elements in the open-source discrete element software YADE. The sphere elements between each cylinder element act like an elbow and thus allow the wire model to bend. The cylinder elements allow for elastic elongation.

Simple models of wire and small sections of wire mesh were created to show that the models can represent the behaviour of high tensile strength wire and wire mesh. The wire model currently does not account for plastic deformation of the wire. However, if the problem to be modelled is primarily concerned with elastic response of the mesh and the complex load transfer mechanisms and deformations of hooked bent wires oriented in different directions, the YADE models appear to do a good job at capturing the phenomena. Simulating the large differences in the strength and stiffness of mesh loaded in the longitudinal versus the transverse direction is an important aspect of the modelling work presented in this paper. A calibrated...
YADE model of the mesh was able to show this behaviour. A rockbolt plate loading test was conducted to show the deformed shape and failure mechanism of the mesh.

This work is the foundation for ongoing research to optimize the design of steep rock slope stabilization with bolted steel mesh systems. The numerical results show that the behaviour of a steel mesh can be well simulated by YADE and this approach can be used as an efficient and effective tool for further studies.

Acknowledgement

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References


Drop testing of concrete panels and welded wire mesh at LKABs Kiirunavaara mine

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B. Krutrök, LKAB Berg & Betong AB, Sweden

Abstract
The mining company Luossavaara-Kiirunavaara AB, LKAB, operates two underground iron ore mines in Sweden. Both mines use large-scale sublevel caving to extract the ore at a depth of nearly one kilometre and are, since 2008, regarded as seismically active. Rock support which is able to resist dynamic loading is therefore essential in order to maintain safe and accessible production areas. The support system used by LKAB primarily consists of steel fibre reinforced shotcrete together with external steel mesh and rock bolts. Accurate methods for design and testing of the support elements are important to ensure stability of the system.

This paper describes dynamic drop tests performed on round determinate concrete panels (RDPs) and welded wire mesh at LKABs Kiirunavaara mine. The test setup, based on the ASTM C1550 standard for static RDP testing, has been designed and evaluated. More than 30 drops have been performed on different types of panels, with impact energies between 1 and 5 kJ. The combination of concrete panels and steel mesh has also been tested. These tests provide a method of comparing the performance of different support elements under repeatable conditions.

1 Introduction
Luossavaara-Kiirunavaara AB (LKAB) owns and operates the Kiirunavaara and Malmberget underground iron ore mines in northern Sweden. Both mines use large-scale sublevel caving to extract the ore and have been in operation for more than one hundred years. The crude ore production from both underground mines was approximately 45 million tonnes during 2015.

Production is currently performed at depths approaching one kilometre. The large-scale mining method used, illustrated in Figure 1, causes significant changes to the in-situ stress state (Sjöberg et al., 2003). At current production levels, the mining-induced major principal stress is approximately twice the magnitude of the in-situ (non-disturbed) stress state and oriented horizontally.

Over time, the requirements of ground support in the mines have changed. Since 2008, the mines have been regarded as seismically active. The seismicity primarily consists of fault-slip type events triggered by the mining-induced stress change. For this reason the rock support standards include support designed for energy absorption, and a cross-section of the support system is shown in Figure 2. Fibre-reinforced shotcrete is combined with welded steel mesh, externally fixed to the shotcrete using yielding rock bolts. For more details about the implementation of the support system, the reader is referred to Jacobsson et al. (2013).
The function of the support system as a whole is highly dependent on the capacity of the individual supportive elements. Accurate and relevant methods of testing support elements are important both in order to design a stable and efficient support system, and to be able to compare alternative products and solutions. In a seismically active mine, both the static load bearing capacity and the energy absorption properties of the ground support requires consideration. While static testing of shotcrete as surface support by the use of round determinate panels (RDPs) is an established and standardized test method, the methods for dynamic tests are not as well formalized. This paper describes the results from a number of drop tests on concrete RDPs and wire mesh, which have been done at LKAB in Kiruna. These results can be qualitatively compared to the standardized quasi-static RDP tests, since the boundary conditions are similar.

2 Sample preparation

The 0.8 m diameter RDPs were cast from the standard recipe used for shotcrete in the Kiirunavaara mine, which yields a cube strength of about 50 MPa after 28 days. More details about the concrete receipt can be found in Thyni (2014). Casting was chosen over spraying in order to obtain more repeatable results, although this results in un-conservative test results compared to the installed ground support. All panels were allowed to cure for about 28 days before testing.
The steel mesh has a wire diameter of 5.5 mm and a wire spacing of 75x75 mm, and was cut to panels of about 1.1x1.1 m. This type of mesh is identical to what is used in LKABs underground mines.

3 Test Method

The test setup was based on the ASTM C1550 (2012) standard for performing RDP tests, except that load was applied by a free-falling drop weight instead of a hydraulic cylinder. The 0.8 m diameter RDP was supported by three ball bearings and transfer plates on a 0.75 m diameter three-legged fixture, as illustrated in Figure 3 and Figure 4. Vertical threaded bars were used to allow testing of concrete panels together with steel mesh, with turnbuckles to tighten the bars against the mesh. Figure 5 shows the fixture with a 1.1 m square welded mesh panel in place, and Figure 6 shows the combination of an RDP and a mesh panel. The drop weight consisted of a 26 kg steel sphere fitted with a threaded bar as shown in Figure 7, enabling the total weight to be adjusted up to 100 kg. A maximum drop height of 7.5 metres was achievable with the available equipment. Compared with the existing testing facilities that have been reviewed by Hadjigeorgiou and Potvin (2011), this set-up is relatively modest in terms of the available energy.

Figure 3 Illustration of the bottom plate of the RDP fixture (dimensions in mm).
Figure 4  Illustration of one of the three identical panel supports (dimensions in mm).

Figure 5  The fixture for testing of concrete RDPs and steel mesh
Figure 6  Combining an RDP with steel mesh

Figure 7  A threaded bar allows additional weights up to a total of about 100 kg

High-speed camera, accelerometers and laser distance sensors were used to monitor the tests, together with manual measurements. The overall instrumentation setup is illustrated in Figure 8.
Note that the positive direction of motion is downwards, which will be followed throughout this paper.

The camera was set up to record at 600 frames per second (fps) at a resolution of 432 x 192 pixels. Deflection calibration was performed with a ruler directly below the impact location. Total deflection and impact duration was measured from the high-speed footage, where impact duration was defined as the time between first contact between the weight and the panel until the velocity of the weight was zero.

An accelerometer was fixed to the drop weight, or the topmost disc. Two types of accelerometers with different measurement ranges were tested; one with 50 g range and one with 5000 g. After low-pass filtering with a cut-off frequency of 1000 Hz, the accelerometer signal was integrated to obtain velocity and deflection of the weight as a function of time.

Continuous deflection measurement was done for some of the tests using two (for redundancy) short-range laser distance sensors, with a typical resolution of less than one-tenth of a millimetre and a measurement frequency of 1.3 kHz. A casing was used to protect the sensors, with mirrors to direct the beams upwards as illustrated in Figure 8. The panel velocity was obtained by numerical differentiation after low-pass filtering with a cut-off frequency of 200 Hz. However, too much noise was present to give a meaningful value of the acceleration as a function of time by differentiating twice.

Since the deflection was measured, directly or indirectly, using four different methods, a weighted average was used to calculate a representative value. The laser sensors were assigned a weight of 40 %, and each of the other three methods a weight of 20 %. These weight were arbitrarily chosen but motivated by the laser sensors being both a direct and continuous measurement method for deflection.

The average and peak forces can be calculated from the impact duration and maximum acceleration, when the mass of the drop weight is known:

---

**Figure 8** Overview of the instrumentation setup. Note that the positive direction of motion is downward.
where:

\[ F = \frac{m \Delta v}{t_{\text{imp}}}, \]

\[ F_{\text{peak}} = m a_{\text{max}}, \]

\( F \) = average force during impact,

\( F_{\text{peak}} \) = peak force during impact,

\( m \) = mass of drop weight,

\( \Delta v \) = change in drop weight velocity before and after impact from accelerometer,

\( t_{\text{imp}} \) = impact duration,

\( a_{\text{max}} \) = peak value of acceleration during impact.

One of the most difficult parts of these tests proved to be the measurements. Consequently, a number of checks were performed to sort out valid data from invalid. For example, accelerometer measurements were discarded if the calculated final velocity of the drop weight differed significantly from zero. As an additional check, the deflection calculated from the double-integrated acceleration was compared to the deflection measured by other methods. Even when the peak acceleration was used to calculate a maximum force, it is questionable if this really represents the peak contact force or just an elastic shockwave through the drop weight. An example of this is shown in Figure 9, where the acceleration is oscillating during the first five milliseconds of the impact.
Results

Drop tests have been performed on 75 mm concrete RDPs with and without steel mesh, as well as on 50 and 100 mm RDPs with steel mesh. Figure 10 shows the deflection obtained at different impact energies for the RDPs with and without mesh. There is more scatter in the test results for the RDPs without mesh than the panels with mesh. The results from Thyni (2014) on statically loaded RDPs combined with steel mesh are also included for reference in Figure 11. An observation from this data is that the kinetic energy that can be absorbed during impact testing is significantly higher than the static energy absorption at the same deflection. A 75 mm RDP absorbs about 0.75 kJ up to 40 mm deflection in the static test, while an impact energy of about 1.25 kJ is required to cause similar deflection in the dynamic case.
Figure 10  Results from drop tests on concrete RDPs with and without steel mesh

![Graph showing results from drop tests on concrete RDPs with and without steel mesh.](image)

Figure 11  Results from static testing of RDPs with and without steel mesh, after Thyni (2014)

There are observable differences in the pattern of cracks depending on velocity. Figure 12 illustrates two typical examples with the same impact energy but different velocities. The panel subjected to a
high velocity impact has a larger number of minor radial cracks than the other panel, as well as a punching shear failure in the centre. This also differs from the static tests, where the panels generally crack cleanly into three pieces (see Figure 13).

![Figure 12](image1.png)  
**Figure 12** Typical examples of cracking of RDPs at the same energy (1.2 kJ) but with different impact velocities

![Figure 13](image2.png)  
**Figure 13** Typical example of cracking of RDP during static testing, from Thyni (2014)

Figure 14 shows at what impact energy failure occurred in the RDPs of 75 mm thickness without steel mesh. Failure is defined either by a total collapse of the panel, or a deflection exceeding 40 mm. This particular value is chosen so as to enable comparisons with the standard quasi-static RDP tests, where the ultimate energy capacity is evaluated at 40 mm deflection. As can be seen, no failure is observed for energies below 1.2 kJ, while all panels above 1.2 kJ failed. There are not enough data points at 12 m/s to determine if the higher impact velocity affects the failure energy. As a comparison, during static testing of 75 mm panels the deflection exceeds 40 mm after about 0.75 kJ.
Discussion

An obvious difficulty when these tests are compared against static tests on RDPs is that no direct measurements of the contact force were performed. However, it is still possible to do qualitative comparisons regarding crack propagation in the panels. The number of cracks developed in the panels are generally higher during impact testing than what is observed for static tests. Also, the pattern of cracking changes significantly at high impact velocities, with a punching shear-type failure under the impact location. More tests are required in order to evaluate what effect this has on the energy absorption properties of the panels.

Although care should be used when drawing conclusions based on the comparison to the static tests, it is notable that more energy is required to reach a given deflection for the dynamic case. However, the impact energy may not be directly comparable to the energy absorbed in static testing, since some part of the kinetic energy is lost as heat (i.e. friction and/or crushing of the panel).

The boundary conditions of the test setup are obviously very different to what the support system is subjected to during a seismic event. Since the RDP is freely supported, there is consideration to the effects of adhesion to the rock surface or clamping by adjacent shotcrete. Furthermore, no attempt has been made to simulate the distributed loading of a broken rock mass. Instead, load is applied by a direct impact on a point on the panel. For these reasons, the quantitative values of energy absorption cannot be directly applied for the purpose of support system design. However, these tests allow for repeatable and comparable testing of different support elements during rapid loading.

6 Conclusions

A number of drop tests on concrete panels have been performed using this test setup, both with and without external steel mesh. The following conclusions are drawn from these tests.

- The results were relatively consistent when the results from a number of drops were compared, especially when concrete RDPs were combined with steel mesh.
• Higher impact energies were required in the dynamic tests to give equivalent deflection of the static tests performed by Thyni (2014).

• There are indications of differences in failure modes of the panels, depending on the impact velocity. However, this requires further testing to verify.

Acknowledgement

The authors would like to thank everyone at LKAB Berg & Betong who helped during the preparation and execution of these tests.

References


Dynamic twisted rockbolt for underground excavation in deep mine conditions

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Abstract

Most of the existing ground support systems have been developed for low and moderately high stress conditions in the rock mass. They utilize either a single type of support element (e.g. exclusively rockbolts or concrete lining, etc.) or a combination of its several types (e.g. rockbolts and shotcrete and/or mesh). It is rather obvious that the existing support systems are not fully suitable to highly stressed rock masses at high depth which favours either squeezing or rockbursts events to occur. In such conditions it is therefore recommended to use ground support systems which is able to deal at the same time with squeezing and burst problems as well. This type of requirements may fulfil only ductile or, in other words, energy-absorbing systems. There is also the industry generated pressure for improvement the existing systems making them more efficient or for development new support systems which could be adapted in squeezing and burst-prone rock conditions. Also there is an industrial need to develop cheaper and still effective ductile ground support systems which could be able to replace the old ones designed years ago and dedicated to mining at rather shallower depths. Generally mines prefer ground support systems which are stiff in the beginning of the load path and become deformable (i.e. ductile) when the load on the support elements exceed a certain limit.

This paper’s objective is to present the idea of development of a new type of the effective and low cost ductile rockbolt system manufactured in different shape, selection of materials enabling sliding of rockbolt along surrounding resin and results of underground pull-out tests involving different bolts’ shapes, diameters of boreholes and different sliding materials set on the rockbolts’ rods.

1 Introduction

A lot of deep mines over the world do utilize ductile elements in their ground support systems. In high stress conditions, these systems revealed to be more effective than stiff ones. The most often applied ductile support types of support includes the following systems:

- Point type bolts - Split sets, Swellex bolts, cone bolts, Durabar bolts, hybrid bolts (Mercier – Langevin and Turcotte, 2007), debonded cables,
- Surface types such as meshes, straps, fibre/mesh reinforced shotcrete and lacing.
- Also, new ductile support elements are under development and/or even after field testing, e.g. Garford dynamic solid bolt in Australia, D-bolt in Norway and Roofex in Austria (Charette and Plouffe, 2007).

Completely new ductile ground support system called “Cuprum Twisted” Rockbolt (CTw-bolt)” has been developed at KGHM CUPRUM, which has filed the patent application to the Polish Patent Office at Jan. 15, 2014 (Pytel, 2014). Some functions and characteristics of the CTw-bolt are similar to those characterizing the tensionable spiral bolt with resin nut patented in U.S. at Aug. 19, 2014 (Fox, 2014). However both bolts have different basic functions since the first one is a typical ductile rockbolt...
while the latter one through “rotation of the spiral portion within the hardened resin nut serves to move the bolt within the borehole, such as for purposes of tensioning” (Fox, 2014).

The CTw rockbolt is linearly anchored in the boreholes filled with cement or epoxy resin. It comprises a smooth, debonded solid steel bar of the rectangular cross-section, twisted repeatedly. After resined in a borehole, CTw-bolt is firmly anchored to the rock along the entire its length. However, when the rock dilates it will load the bolt shank which is pulled out overcoming the resistance generated by the steel rod. The rod’s ability to resist depends mostly on the number of the rod coils and the pitch size. The total elongation of the CT-bolt is expected to be considerably large.

2 Design of CTw-bolt

The twisted rockbolt induces the friction load as well as plastic deformation in 3-dimensional pattern, due to providing the appropriate spiral shape of the rod (cylindrical or conical type of the spiral) with different diameter and coils. Absolute novelty system requires boreholes of small diameters (up to 24 mm). Very effective mixing of 2-element resin resulting in exceptionally tight borehole filling.

Figure 1 New ductile ground support system – Cuprum Twisted rockbolt

2.1 Twisted rockbolts’ prototypes

The prototypes of CTw rockbolts have been manufactured of grade III structural steel 34GS (U.T.S. = 600 MPa and yield stress Y.S. = 420 MPa), according to the drawing presented in Figure 2. They were ordered in five types which differed mutually by number of coils: 0, 0.5, 1, 2 and 4 coils per meter. Since the primarily designed length of 2.0 m has revealed to be too great - the 5 first tested rockbolts’ being subjected to the maximum capacity of the used hydraulic jack (15 t) have not displayed any signs of movement - for safety reason, all remaining bolts have been shortened to 1.6 m.

The ultimate tensile capacity of the rockbolt has been has been evaluated as $Q_{ult} = 15.3$ t.
Figure 2  Drawings of the twisted rockbolts prototypes
2.2 Sliding coats applied for friction and adhesion reduction on the resin – rockbolt contacts

The tested rockbolts have been covered by different types of sliding substances. Most of them are made of polymers, some of them reinforced by ceramic additives.

Table 2 Types of coating used in field tests

<table>
<thead>
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<th>Type of coat</th>
<th>Coat characteristics</th>
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<tr>
<td>RC 25,4/12,7</td>
<td>Heat shrink tubes by RADPOL are made of polyolefin (e.g. polyethylene).</td>
</tr>
<tr>
<td>Powder coat</td>
<td>Coating based on the polymers PTFE containing the resins improving the abrasion resistance as well as providing better sliding by the effect of dry lubrication. This coating reveals extremely high corrosion resistance. The film thickness is of about 25 µm ± 5 µm.</td>
</tr>
<tr>
<td>N 456</td>
<td>Coating based on the PTFE, PFA, FEP polymers mixture and additionally reinforced by ceramics. It reveals enhanced non-stick properties and the high abrasion resistance. The film thickness is of about 40 µm ± 5 µm.</td>
</tr>
<tr>
<td>NI 4611</td>
<td>Fluoro-polymer coating based on the polymers FEP. It has a very smooth surface of the high non-stick and chemical resistance parameters.</td>
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<tr>
<td>NWP 058</td>
<td>Coating based on the polymers FEP displaying high abrasion resistance and the high non-stick parameters. The film thickness is of about 25 µm ± 5 µm.</td>
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<tr>
<td>NW118B</td>
<td>Coating based on the polymers PTFE reinforced by resins. The purpose of the coating is to reduce friction and to protect against seizing. It is wear off resistant and moderate non-stick properties. The film thickness is of about 20 µm ± 5 µm.</td>
</tr>
<tr>
<td>NW 101</td>
<td>The values of friction coefficient CoF: coating-steel for the N456 and NW101 sliding surfaces varies from 0,02 to 0,05.</td>
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</table>

3 Field rockbolts pull-out tests

The CTw rockbolts’ prototypes were tested within the panel G-11 located in the western part of Rudna mine (Figure 3), one of the KGHM’s copper mines in Poland, at the depth of 1 130 m. The chosen site geology was typical for the area with immediate roof strata composed of relatively strong rocks such as dolomites and limestones (see Figure 4). Their thickness did not vary significantly over the area. Roof strata strength parameters differ significantly from the rocks located in a lower horizon, particularly from those composing relatively weak floor strata complex.

As a matter of fact, such geotechnical conditions are very advantageous from point of view of rock mass stability potential and therefore almost exclusively the rockbolt systems are applied as a ground support: mechanical expansive shell bolts, resin rebars, cable bolts and friction bolts. The total number of used rockbolts is greater than 2.25 million sets a year.
Figure 3  Location of research site within the vicinity of R-IX area of Rudna Mine

Figure 4  Rock mass properties determined from borehole Mo9 451 cores in Rudna mine
The static pull-out force has been generated by hand-pumped Enerpac hydraulic jack set of 15 t nominal capacity (Figure 5). Results of 35 tests are presented in table 2.

Figure 5  Installment and pull-out tests of the rockbolt prototypes

The principal method applied for rockbolts performance testing is a laboratory pull-out testing. The basic procedure of all the variants of pull-out test is the similar: after installing a bolt is being pulled out until failure event occur and the ultimate load is registered as the load capacity of the bolt. The most simplest small-scale test method relies on installing the bolt in two steel cylinders and afterwards the cylinders are pulled apart from each other until failure occurs. Full-scale tests however, large concrete blocks engage to simulate “rock mass” tests (Stillborg, 1994). In the most of laboratory and field tests, the load applied was of the axially directed static and tensional type of load. However, in some, rather rare cases, a lateral static load has been applied during the shear tests (Stjern, 1995). The number of publications dealing with static pull-out tests conducted in laboratories is relatively high, e.g Dodd oraz Fasught (1973), Ludvig (1983), Stillborg (1994), Stjern (1995), Dahle i Larsen (2006).

However, static tests, laboratory and field conducted, deal with relatively slow process of load increment which in the appropriate way represents (a) processes associated with progress of mining visualized mostly by changes in geometry of the excavated area, (b) squeezing of soft and plastic geological structures deforming by the almost constant load. Steel pipes analogues utilized in laboratory tests due to their high suppleness may however create in-correct results. Therefore the well-designed test facility should has the exceptionally stiff structure. In a case of field pull-out measurements one should agree that kind of tests is the most suitable and do not rise any provisions in the practical usage. Thus it seem to be justified opinion that replacement of field tests by the laboratory tests may take place only within the limited areas (introductory, provisional tests) since
persistent modeling of the natural mining/geological conditions in the laboratories, usually reveals to be economically and technically unjustified.

Table 2  Results of pull-out tests

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Rockbolt notation: 1 – 0 coils, 2 – 0.5 coils/m, 3 – 1 coils/n, 4 – 2 coils/m, 5 – 4 coils/m

Since the performed pull-out tests have not engaged any instruments which would be able to relate precisely the acting load and the corresponding displacement of the rockbolt, the authors have relied only on the manometer indications and the visual observations manifested by the pullling-out element’s behaviour. Therefore the obtained results include two basic load values assigned to the two consecutive phases of loading (see also Figure 6):
- adhesion breaking point – the maximum load carried out by the rockbolt at the very beginning phase of deformation – the corresponding elongation of the rockbolt is no more than 2 mm, and
- friction resistance – the almost constant pulling-out load associated with the displacement up to 20 mm.

Obtained results are – in fact, preliminary, however some significant conclusions may be drawn. Results of performed pull-out tests indicate their strong relationship on the diversity of sliding layer’s characteristics which have been actually applied. From the seven series of tests, Serie 6 has produced results (Figure 7) which seem to be the most rational in respect to the accepted assumptions and expected technical behaviour.

![Figure 6](image1.png)

**Figure 6** Definition of adhesion breaking load and pull-out load characterizing the field tests

![Figure 7](image2.png)

**Figure 7** Tests results performed for Series 6 N456 coating

### 4 Novel laboratory pull-out tests facility

Further investigations of designed CTw-bolts are planned to be focused on static and dynamic pull-out tests conducted in laboratory conditions, showing the complete pull-out load vs deformation characteristics.
During the last decades, several facilities for rockbolts’ dynamic pull-out testing have been developed in laboratories, in Canada (Charette and Plouffe, 2004; Plouffe et al., 2008), Australia (Player et al., 2004, cit. Hadjigeorgiou, 2007), RSA (Ortlepp & Stacey, 1998), Switzerland (Cała et al., 2013). All of that facilities utilize similar mechanical principles – gravitational or hydraulic driven weight dropping from a given height resulted in the impulse type of load acting on the rockbolt stabilized in a borehole imitation, most often made of thick-walled steel pipes. Unfortunately, the impulse load generated by the drop weight facility is not necessarily similar to the load which rockbolts are subjected to in the underground natural conditions.

According to the knowledge of the authors, there has been no information on a rockbolt testing facility which would be able to simulate time-dependent dynamic load similar to that produced by induced seismic events. This testifies innovation of the proposed method and its uniqueness.

Conducting research in the natural mining conditions always seems to be the appropriate solution for increment of the knowledge about a performance of different types of rockbolts. However, the hydraulically operated prototype test facility manufactured in the framework of the I²Mine project, will make it possible to test in laboratory the CTw-rockbolt construction in order to verify their suitability for use in the presence of dynamic type of rock mass pressure and their appropriate selection according to the local mining-and-geological conditions (Madziarz et al., 2014).

The thick-walled steel cylinder of diameter of 400 mm is the basic part of the facility (Figure 8). At the one of its end the fixing unit is attached while at the other end the hydraulic jack is located (Figure 9). Basically the developed test facility is hydraulically driven by the hydraulic jack which piston is the only moveable element of the apparatus. It is generally a one-degree of freedom mechanism in which the oil pressure within jack is transferred onto the rockbolts’ plate and nut. Finally the load from the nut is fully transferred onto the rod of the rockbolt. The hydraulic supply unit is composed of the oil container on which the pumping unit, hydraulic accumulator, elements of control and auxiliary elements are located. The pumping unit is combined of electrical multi-piston hydraulic pump of smoothly controlled output. The hydraulic accumulator will be switched on when the pressure demand will be larger than the pump actual capacity. Furthermore, for maintaining the appropriate temperature of the flowing oil, an independent cooling unit is provided in the developed facility.

Figure 8 The concept of the facility for mechanical bolts’ testing

One of the most important feature of the facility is a very high stiffness of the corps what permits assuming that practically all deformations generated by hydraulic jack will be transferred exclusively onto the tested rockbolts. A such clearly defined behavior is of very high importance, particularly when high values of acceleration induce the inertia forces within the facility elements’ masses. The test facility permits a continuous measurement of time-dependent static and dynamic load applied on the rockerbolts and generated by the excitation generator.
The test facility has also been modeled based on 3D finite element method using the FEMAP (2008) and NEiNASTRAN (2001-2009) computer code. The technical drawing allowed to create, in the numerical space, the true copy of the considered apparatus (Figures 10).

The external load has been modeled as the uniaxially time-dependent force acting along the tested rockbolt: Run No. 1 – translational load as in Figure 11 (damping coefficient $\rho = 0.05$), and Run No. 2 – translational load as in Figure 11 (damping coefficient $\rho = 5.0$).

The calculation time step has been assumed to be $\Delta t = 0.001$ sec. The preliminary computations have also revealed that the change $\Delta p = 1 MPa$ of the pressure acting on the jack’s piston induces the tensile stress increment within the rod of the rockbolt (16x16 mm) up to $\sigma = 23.52 MPa$ as well as the length increment equal to 0.000206 m. The knowledge of these (sensitivity) parameters of the loaded system have permitted for full controlling of oil pressure within the jack and maintaining the desired displacement (Figure 11) of the nut and plate. Selected computation results are presented in Figure 12.

![Figure 9](image9.png)  
**Figure 9** Longitudinal cross-section of the driving hydraulic jack

![Figure 10](image10.png)  
**Figure 10** General view of the facility FEM model (left) and cross-section through the hydraulic jack (right)
Figure 11  Vibration of the expansion shell rockbolt supporting a detached 4t rock block during seismic deformation coming from the surrounding rock mass (Fabiańczyk, 2016)

Figure 12  Principal stress $\sigma_1$ (left) and $\sigma_3$ (right) contours (MPa) obtained for time $t = 0,594$ sec, within the longitudinal cross-section of the facility corps (Run No. 1).

Figure 13  Contours of displacement $w_x$ (m) values for time $t = 0,594$ sec, within the longitudinal cross-section of the facility corps (Run No. 1).

The conducted numerical analyses of functionality of the test facility have reveal that:

1. The facility structure may be, with ample reserve, subjected to the load greater that strength of the steel rod of the diameter of 32 mm.
2. Due to the massive facility corps, its deformations during operation are negligible small. Therefore one may assume that the proposed test facility belongs to the group of stiff appliances.

3. The computer simulations also revealed that the load generated within the facility remains in equilibrium and do not affect the elements of foundation. The support reactions are induced by nothing except the facility weight.

4. Within the dynamic load frequencies’ range \( f = 2 \text{–} 150 \text{ Hz} \), a significant distortion of the hydraulically generated signals due to induced inertia loads, is not observed. Taking into account the nature of the phenomenon, using frequencies higher than 150 Hz seems to be inappropriate.

5. The developed test facility permits testing all kind of rockbolts (resin, mechanical and friction types) working under any regime of kinematic excitations. This proves the uniqueness and usefulness of the developed facility.

5 Conclusions

The obtained results of pull-out tests have proved their usefulness in the Polish copper mines conditions as an effective mean for ground support reinforcement. However due to a specific field conditions and some limits of the equipment applied, results are not complete and may be treated as a preliminary before full pull-out tests with displacements up to 500 mm could be conducted in laboratory. Nevertheless the results include the values of two basic parameters:

1. Critical load – maximum pull-out load maintained by the rockbolt – which is a sum of the adhesive strength and friction mobilized on the surface of rockbolts.

2. Residual load – exclusively equal to the frictional effects.

The pull-out load values presented in table 2 indicate their strong dependency on the kind of sliding coat’s characteristics which have been actually utilized. Although the available results are rather of the preliminary nature, nonetheless from them one may already draw some important conclusions, among them:

3. Heat shrink tubes provide relatively low values of the pull-out forces.

4. Number of coils from 1 to 2 per meter seem to be the most rational. Series No. 6 offers very reasonable results from this point of view.

5. The rockbolt load capacity may be also controlled by its length.

Furthermore, since the proposed ductile twisted bolt system requires smaller diameter of the borehole and characterizes itself by the exceptionally simple structure, it is potentially predestined to be a good candidate for replacement of some existing, more complicated and more expensive ductile rockbolt types utilized presently in mines. Against the characteristics of the most aforementioned systems, the proposed ductile CT-bolt system seems to manifest some advantages since:

6. It needs narrow boreholes (D25 mm) - shorter time of development and less energy necessary to use,

7. Its construction is simple and inexpensive to manufacture, and

8. Owing to its simplicity it is expected to behave well even in complex strain conditions.
Acknowledgement

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Establishment of experimental sites in three Swedish mines to monitor the in-situ performance of ground support systems associated with mining-induced seismicity

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Abstract

In order to assess the performance of ground support components and systems when subjected to seismic activity and strong ground motion, Luleå University of Technology together with three Swedish mining companies (Lundin Mining, LKAB and Boliden) started a three year research project in September 2014. The aim of the project is to develop new methods for evaluating the rock support performance in-situ that use all available information about i) the source of the seismic event (obtained from the seismic network in the mine and additional seismic sensors), ii) seismic loading (ground motion) recorded by temporary local seismic networks, and iii) the consequences of the seismic loading in terms of damage to the underground excavations and the rock support.

The sites with high potential of seismic damage were defined after the historical damaging seismic events were reviewed and the mining-induced stress disturbance was investigated with 3D numerical models. As of 31 December 2015, four sites in three different mines have been instrumented. Geophones (in depth and at surface), multi-points extensometers and instrumented bolts were installed to monitor the ground motion, the deformation of the rock mass and the elongation of the bolts. Observation boreholes were drilled to investigate the rock lithology, structures as well as fracture distribution and development. The data from locally installed geophones will be integrated with seismic data recorded by the mine-wide network. For each monitoring point, all of the instruments and observation boreholes were located at very close area within 0.5-1 m distance from each other. These results will be used to establish the relationship between the dynamic loading and the response of rock mass and rock bolts. Additionally, laser scanning is used to measure the surface deformation of the whole volume of instrumented sites with time. Two damaging seismic events occurred near the instrumented sites after the instruments were installed and the results of site investigation show that installed instruments have captured the response of the rock mass and bolts due to production blasting and seismic events.
1 Introduction

Safe and stable underground constructions are crucial to achieve optimal utilisation of mineral resources and efficient mining at great depth. The escalation of ground control problems as well as the increased occurrence of damage caused by seismic events with increasing magnitude jeopardize the safety and may lead to injuries of the personnel, damage to equipment, ore losses and unplanned operational disturbances. One of the prime concerns for deep mines is the issue related to the performance of the ground support under these conditions.

In order to assess the performance of ground support components and systems when subjected to seismic activities and strong ground motion, laboratory tests on cores, drop tests, simulated rockburst experiments and back analysis for case studies have been employed in different mines. However, none of these techniques are successful due to different limitations (Hadjigeorgiou and Potvin, 2007). The most realistic assessment of ground support systems to dynamic loads is the active monitoring of real seismic events and the corresponding damage. Unfortunately, as these events are unpredictable in time and space, the sites where the events occur are rarely adequately instrumented “a priori”. The design and selection of data for various support components would be improved if quantitative measurements were made particularly in the vicinity of large seismic sources (Ortlepp, 1984). Furthermore, many direct relationships between damage levels caused by rockbursts to the rock and ground support around an excavation and ground motion parameters have been developed (e.g. Kaiser et al., 1992; Heal et al., 2006). However the applicability of these relationships has been limited due to difficulties resulting from highly subjective assessment criteria and incomplete or questionable peak ground motion data. This limitation will only be overcome when high quality strong motion data has been collected and properly analysed (Jesenak et al., 1993). Even though seismic monitoring systems have been installed in most of the deep mines, there is still lack of systematic active monitoring and integrated information obtained from different types of geotechnical instrumentation thus resulting in difficulties to identify the seismic event mechanisms, to understand the damage process of the rock and support system, to judge the support effectiveness and to understand the interaction between the rock and the rock support during a seismic event.

Luleå University of Technology together with three Swedish mining companies (Lundin Mining, LKAB and Boliden) started a three-year research project in September 2014. The project aims at developing new methods for Evaluating the Rock Support Performance (ERSP) in-situ that use all available information about i) the source of the seismic event (obtained from the seismic network in the mine and additional seismic sensors), ii) seismic loading (ground motion) recorded by temporary local seismic networks, and iii) the consequences of the seismic loading in terms of damage to the underground excavations and the rock support, in order to find reliable relationships between the seismic event and the factors affecting the damage to the underground excavations and the rock support and in this way to reduce the number of production disturbances caused by poorly understood rock mass and rock support response, thereby decreasing the risk for personnel injury and production losses. The methodology developed for this project, the description of different types of monitoring equipment and systems, as well as the first results from the monitoring are given here.

2 Field instrumentation

2.1 Methodology

One of the main ideas in the project was to install different types of instruments in drifts/stopes where there is a high likelihood that damage due to seismic events will occur in each one of the mines: the Zinkgruvan Mine, the Kiirunavaara Mine and the Garpenberg Mine, Sweden and to conduct four different measurements at each site: i) transient response of the rock at excavation surfaces and at depth as a result of seismic impingement, ii) permanent displacement changes and fracture development around excavations, iii) deformation of excavation surface, and iv) deformation development along rock bolts. Different types of monitoring data have to be integrated to establish relations between the parameters of
the seismic source/seismic waves and the performance and damage of the supported excavation (rock mass and rock support). The monitoring is complemented by numerical analyses and further forensic investigation after a seismic event has occurred.

The generic procedures followed at the experimental sites and the key instruments for field monitoring are listed below.

- **Selection of experimental sites.** In 2014, several project meetings were organized for selection of the experimental sites in the three mines. Seismologists, geologists and rock mechanics engineers were participating at these meetings to identify sites that will have high likelihood of rockburst damage during the project period 2015-2017 and beyond that time period. Preliminary numerical analyses of global models (mine scale) and analysis of past seismic data were carried out to identify areas where significant mining-related seismicity had been observed in the past and could be expected in the future. Furthermore, some criteria were defined in order to satisfy the installation requirements from a practical point of view and provide enough data within the project period. These criteria were that the sites should i) have access to services (electricity, water, compressed air, etc); ii) be far from source of mechanical noise (e.g. ventilation fans, orepass, etc); iii) be preferably recently developed and hence the instruments could capture most of effects from subsequent mining activities; and iv) be accessible as long as possible to facilitate collection of as much data as possible. After the discussions, underground visits were made to ensure that the sites met all criteria.

- **Multiple Point Borehole eXtensometers (MPBXs):** Deformation monitoring in the rock mass around an excavation at 6 discrete measurement points is carried out by using MPBXs. Therefore, the deformation development within the rock mass surrounding the excavations is evaluated before and after a seismic event.

- **Laser scanning:** Deformation of the surface of the test site is monitored by using laser scanning. After the initial scanning, the time for conducting a second scanning should be determined when the anchor of the MPBX located closest to the surface has detected deformation of more than 10 mm.

- **Instrumented bolts:** The deformation development at 6 distributed segments along the bolt is investigated by using instrumented bolts. The corresponding load development along the bolt is indirectly assessed later when the deformation is within the elastic limit of steel.

- **Borehole observation:** The holes were drilled near the other instruments and surveyed using a slim borehole camera to identify the lithology, discontinuities and changes in the rock mass (e.g. fracture development) due to mining activities and seismic events.

- **Local seismic systems:** Data from seismic sensors installed near and around the excavations of the test sites provide input for analyses of the wave field close to the excavations. This data is used to evaluate the local variations and site effects on the ground motion; and together with the data from the mine wide seismic monitoring system it is used to evaluate the seismic wave attenuation and establish the relationship between the seismic source parameters and the rock mass and rock support damage due to a seismic event.

- **Damage mapping:** Damage mapping should be conducted at the experimental sites immediately after a seismic event has occurred according to the developed forensic manual. The work should be mainly conducted by a research team with additional experts (e.g. geologists, seismologists and rock mechanical engineers) involved.

- **Numerical modelling:** Numerical models at local (e.g. drift) scale with detailed support information are built and numerical analyses will be carried out to investigate the response of the supported excavation under static loading (e.g. mining advancing) and dynamic loading (e.g. seismic event) conditions. The monitoring data is used to calibrate the numerical models.
Data recording (e.g. format, frequency), preliminary analysis, archiving procedures and maintenance schedules have been established at the beginning of this project. All data from the monitoring is also used as additional information to establish the relationship between the seismic event/seismic waves characteristics and the rock/rock support damage. Table 1 shows the summary of the experimental sites as of 31 December 2015. Figure 1 shows the schematic layout of instrumentation and instrumentation array at one of the experimental sites at the Kiirunavaara underground mine.

### Table 1  Summary of the experimental sites as of 31 December 2015

<table>
<thead>
<tr>
<th>Mine name</th>
<th>Zinkgruvan</th>
<th>Kiirunavaara</th>
<th>Garpenberg</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Basic information</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Owner</td>
<td>Lundin Mining</td>
<td>LKAB</td>
<td>Boliden</td>
</tr>
<tr>
<td>Orebody at experimental site</td>
<td>Burkland</td>
<td>Kiirunavaara</td>
<td>Lappberget</td>
</tr>
<tr>
<td>Mining method</td>
<td>Sublevel stoping (Underhand mining)</td>
<td>Sublevel caving</td>
<td>Sublevel stoping</td>
</tr>
<tr>
<td>Depth at experimental site</td>
<td>1150 m</td>
<td>1108 m</td>
<td>728 m</td>
</tr>
<tr>
<td>Concern</td>
<td>Rock support for underhand mining</td>
<td>Rock support at seismic active areas</td>
<td>Rock support in sill pillar</td>
</tr>
<tr>
<td><strong>Ground support at experimental site</strong></td>
<td>Site 1</td>
<td>Site 2</td>
<td></td>
</tr>
<tr>
<td>Surface support</td>
<td>Fiber reinforced shotcrete</td>
<td>Fiber reinforced shotcrete</td>
<td>Fiber reinforced shotcrete, welded steel mesh</td>
</tr>
<tr>
<td>Reinforcement</td>
<td>NMX-Dynamic bolt</td>
<td>Fully grouted rebar</td>
<td>D-bolt + NMX-Dynamic bolt</td>
</tr>
<tr>
<td><strong>Instrumentation</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>MPBX</td>
<td>16</td>
<td>16</td>
<td>17</td>
</tr>
<tr>
<td>Instrumented bolt</td>
<td>7 (NMX)</td>
<td>8 (Rebar)</td>
<td>7 (D-bolt)</td>
</tr>
<tr>
<td>Laser scanning</td>
<td>5 times (planned)</td>
<td>5 times (planned)</td>
<td>5 times (planned)</td>
</tr>
<tr>
<td>Local seismic monitoring system*</td>
<td>5U+3T</td>
<td>7U+3T</td>
<td>12U+4T</td>
</tr>
<tr>
<td>Observation boreholes</td>
<td>21</td>
<td>22</td>
<td>34</td>
</tr>
</tbody>
</table>

*U indicates uni-axial geophones and T indicates tri-axial geophones.
2.2 Instruments and site installation

To fulfill the requirements of this project, almost all of the geotechnical instruments were improved and slightly modified from their commercially available version before purchasing.

2.2.1 Multiple Point Borehole eXtensometer (MPBX)

The MPBX from YieldPoint (Canada) was chosen in this project after market survey and discussion with the three participating mines by considering the cost, service, users’ evaluation and technical flexibility. As the purpose of this project is to assess the response of the rock mass and ground support system under seismic conditions, there is high likelihood of rock fall or rock ejection near the excavation surface at the experimental sites due to possible seismic events. Therefore, the MPBX was specifically manufactured in the way that the head of the MPBX could be installed at the toe of the borehole to avoid complete loss of data due to potential rock ejection near excavation surface (collar of borehole) (YieldPoint, 2014). The lead wire was then protected by using a steel tube passing from the toe to the collar of the borehole. A trial installation was conducted in one of Boliden’s mines before formal installation of all MPBXs to investigate if there was any potential installment problem. The trial showed positive results, which proved the feasibility to use head-at-toe configuration.

MPBXs with 6 m length and 6 anchors were used in all three mines. However, different anchor locations were chosen for different mines. The anchor locations were chosen in the way that the deformation near excavation surface especially in the fractured zone can be monitored and the deformation in the rock can be compared with the deformation in the instrumented rock bolt at approximately the same depth. As the three mines used different bolt types and bolt lengths, the anchor locations for MPBX were different, see the anchor locations for the designed MPBX used at the Garpenberg mine in Figure 2 (a).

2.2.2 Instrumented bolt

A new rock bolt instrumentation strategy has been proposed and implemented by YieldPoint based on an array of sub-micron resolution displacement sensors that can measure the change in displacement or
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stretch of the bolt. The long strain-gauge based bolt instrumentation appears to provide a satisfactory performance while covering almost the entire length of the bolt based on case studies (Spearing et al., 2013). Therefore, all of the instrumented bolts used in this project were manufactured by YieldPoint.

In order to control the unit cost, the number of displacement sensors for each instrumented bolt was limited to six. The displacement sensors were arranged with three in each diametrically opposed slot (sides A and B of the rock bolt) in an end-to-end arrangement. Sensors denoted by 1, 3 and 5 are on side A and sensors 2, 4 and 6 are on side B. Two different configurations referred to as i) stacked and ii) staggered were used. For the D-bolt and the NMX-Dynamic bolt a stacked configuration was used, while a staggered configuration was used for the fully grouted rebar. A staggered configuration used for the instrumented rebar bolt at Garpenberg underground mine is shown in Figure 2 (b).

To minimize manual work, the data from the MPBXs and the instrumented bolts are recorded automatically by using d4 LOGGER powered by four 1.5V batteries. The reading rate is set as 1 reading per 2h to capture the deformation change in a small time interval which is important in this project especially when a seismic event occurs.

(a)  (b)

Figure 2  (a) Anchor locations of the MPBX, and (b) the sensors distribution along the instrumented rebar bolt used at the Garpenberg underground mine.

2.2.3 Laser scanning

Field observations have shown that the deformation distribution around an excavation periphery is not uniform and localized deformation and failure has often been observed. By using 3D laser scanning, the deformation distribution of the rock surfaces of the experimental site can be obtained. However, the accuracy of measured deformation by using laser scanning is not as high as MPBX. The accuracy depends upon several factors, including the resolution and accuracy of the scanner, the accuracy of surveying targets, and the method and parameters to calculate the deformation (Feng and Röshoff, 2015).

It was therefore decided to use MPBX together with laser scanning to monitor the surface deformation of the excavation. The experimental sites were planned to be scanned at irregular periods. After the initial scanning, the time for conducting a second scanning is determined by means of the deformations measured by the MPBX, i.e., when the anchor located closest to the surface detects a deformation more than 10 mm.

2.2.4 Borehole camera

The first objective of using the observation holes is to identify the lithology and discontinuities at the experimental sites when they are scanned at the initial stage. The second objective is to investigate the changes in the rock mass (e.g. fracturing) as well as the development of the fractured zone with respect to extension and position due to mining advance and seismic events. Finally, the results of these observations will be used to build relationships with the monitoring results from the MPBXs and the instrumented bolts nearby. Therefore, for each monitoring point, all of the instruments and observation boreholes were located within 0.5 - 1 m distance from each other. It was decided to use a bolt rig to drill the observation holes. This means that the borehole diameter is around 26 - 38 mm which requires a small borehole camera.

The Slim Borehole Scanner (SBS) manufactured by DMT, Germany, can produce a 360° digital optical scanning of the borehole wall and was purchased with special requirement for this project. SBS has a diameter of 23 mm and can scan up to several meters long boreholes with the aid of connected pushing rods. With the DMT analysis software, the images can be used for lithological analysis, geological structural
analysis (e.g. bedding, foliation, joints, faults) and geotechnical analysis (e.g. fracture development) (DMT, 2014). The special requirement placed in the order is to improve their current depth measurement system in order to reach higher accuracy and facilitate field operation.

2.2.5 Local seismic monitoring system

The local seismic monitoring systems were installed at the same sites as the MPBXs, instrumented bolts, and observation holes. The sections in which the seismic sensors (seismic profiles) were installed (are up to ~50 cm from the other instruments. In each profile one seismic sensor is installed in the roof and two more in the walls, except at the intersections where one sensor is installed in the roof and/or one in the walls (see Figure 1 (b)).

The local seismic systems installed at the instrumentation site (not the mine wide system) consist of uni-axial and tri-axial 4.5 Hz geophones. The signal from the sensors is digitized with frequency of 6 kHz (Kiirunavaara Mine) or 12 kHz (Zinkgruvan and Garpenberg Mines). Data is collected and transferred by IMS (Institute of Mine seismology) to the server on the surface. The timing of the seismic system is either synchronized with that of the mine wide seismic systems (Garpenberg and Kiirunavaara Mines) or Internet synchronized (Zinkgruvan Mine). The systems run mostly in triggered mode, with remote access to the data. The software for data processing is provided by IMS.

Most of the sensors (geophones) are uni-axial, installed either directly on the surface of the walls after removing the shotcrete (Zinkgruvan Mine) or in shallow boreholes, 50 - 60 cm from the surface (Garpenberg and Kiirunavaara Mines). The sensors are sensitive in a direction perpendicular to the surface of the openings. At every site/mine there are two sets of tri-axial sensors, each set consisting of one sensor installed at shallow depth and one installed in a deeper borehole 6 to 9 m. This was done to study the effect of the free surface/opening on the seismic wave characteristics (amplification, reflection, etc.). After installation and recording it was found that the clipping level of the geophones (< 10-20 cm/s) was reached for some seismic events and it was decided to decrease the gain of a few sensors at each site and increase the clipping level to be able to record larger particle velocities.

The aim of the seismic systems is to record the seismic events in the vicinity of the installed rock mechanics instruments to provide information about the dynamic response of the excavation to seismic wave loading. Furthermore, the seismic instrumentation will also provide valuable data about the seismic waveforms in close proximity to the underground openings with the variations in small distances and between the walls and the roof. Data is used: i) to study near-surface effects, the site effect on the amplitudes, frequency content, and duration of the seismic signals, ii) to study the attenuation/amplification of the seismic waves close to the underground openings, iii) to study the effect of the radiation pattern of the seismic source and the wave propagation effects, and iv) for comparison with the signals recorded by the extensometers and instrumented bolts in case of larger seismic events and rockbursts and blasting in the surrounding area. The final aim is to obtain new information that can be used for improved requirements for the rock support in rockburst prone areas.

3 Experimental sites

3.1 Zinkgruvan Mine

3.1.1 Site description

The Zinkgruvan underground mine is located approximately 200 km southwest of Stockholm in south-central Sweden. The mine has been in continuous operation since 1857, producing zinc-lead-copper-silver ore (zinc as the primary metal) by using underground mining methods. Lundin Mining purchased the mine from Rio Tinto in 2004 (Lundin Mining, 2015).

The experimental site is located at the 1150 m level in the Burkland orebody. Between -1125 and -1300 m (175 vertical metres), this orebody ranges in thickness from 5 to 45 m (average 25 m) over a strike length of
The orebody dip ranges from 55 to 85 degrees over this depth interval. The mine uses longhole primary/secondary panel stoping in the Burkland orebody. Recently underhand panel stoping has been introduced to the lower sections of the orebody. All stopes are backfilled with either paste tailings and cement or waste rock. The current primary and secondary stopes are 20 m high and 15 m wide. They are accessed from the footwall through a decline ramp with dimensions 5 m x 5 m. The ore is stratiform and occurs in a metatuffite with intercalated beds of marble, dolomite and fine grained quartzite. These beds give the ore body a distinctive stratification and significantly reduce the rock mass strength where they are abundant. The footwall rocks are generally competent and massive siliceous metatuffites (leptites). The country rock in the immediate hanging wall of the ore zone consists mainly of calc-silicate bedded metatuffite. This consists of alternating 0.5 cm to 1 cm thick layers of quartzite, quartzitic metatuffite and other metamorphosed rocks, which tend to accentuate the bedding and create ground control problems.

The experimental sites are located in the secondary stopes (S871 and S874). The support system at the experimental sites consists of 55 mm fibre reinforced shotcrete (40 kg/m³ steel fibre), 70 mm wide straps, 2.3 m long 25 mm diameter resin grouted rebars spaced on a grid of 1.2 m x 1.2 m and cable bolts with 2.0 m spacing and various lengths (7 m at sidewall, 6 m in the roof and 10-12 m at the shoulder). The details of the instrumentation at the experimental site can be seen in Table 1.

### 3.1.2 Monitoring results

Seven instrumented NMX-Dynamic bolts, sixteen MPBXs and eight geophones were installed at site 1 (S871) in May, 2015. Eight instrumented fully grouted rebar rock bolts, sixteen MPBXs and ten geophones were installed at site 2 (S874) in July, 2015. Until now, very small deformation has been observed in the rock mass and the instrumented bolts. However, a damaging seismic event with local (IMS) magnitude of 0.9 occurred during production blasting and produced damage along the footwall drift near the experimental sites on July 2, 2015, see Figure 3. One of MPBXs closest to the source at site 1 (S871) shows a displacement jump in Figure 4 (a) after the production blasting and the seismic event. It seems that all of the anchors (located within 4.3 m distance) were affected by the blasting and the seismic event. Comparison of ground motion in the solid rock (at 9 m depth from surface) and at the excavation surface at the experimental site 1 shows site amplification of particle velocity in the fractured zone near excavation surface (see Figure 4 (b)).

Until now, it is not clear if the damage and site amplification of particle velocity were caused by the blasting or the seismic event as the seismic event occurred during the blasting period and the detailed seismologic and rock mechanical analysis is ongoing. However, the occurred damage proves that the experimental sites are located at the right place where high possibility of rockburst could be expected during the project period.
Figure 3  (a) Location of experimental sites and damage distribution along the footwall drift, and (b) location of the blast hypocentre and the seismic event hypocentre on July 2, 2015 at Zinkgruvan mine (The red lines show the ray paths between the hypocentres and local seismic sensors.)

(a)  (b)

Figure 4  (a) Monitoring results from a MPBX, and (b) site amplification of particle velocity during the blasting and seismic event recorded at the roof surface and at 9 m depth in the roof.

3.2 Kiirunavaara Mine

3.2.1 Site description

The Kiirunavaara underground mine is located in the city of Kiruna, approximately 150 km north of the Arctic Circle in northern Sweden. The mine, owned and operated by LKAB, has been in operation since 1898 and produces iron ore. The mining method used in the mine is sublevel caving (LKAB, 2015).

The orebody is tabular and more than 4000 m long, striking almost north-south and dipping 55°-60° towards east. The width of the orebody varies from a few meters up to 150 m, but averages around 80 m. The experimental site is located at the 1108 m level and the instruments were installed along the footwall drift between y-coordinates Y32 and Y33. The main host rock type in the footwall drift is Precambrian aged trachyo-andesite (Henry and Dahner-Lindkvist, 2000), which is internally designated as syenite porphyry.
Establishment of experimental sites in three Swedish mines to monitor the in-situ performance of... P. Zhang, et. al.

and subdivided into 5 categories with uniaxial compressive strength varying between 140 - 300 MPa (Sjöberg, 1999).

The support system which is used at the experimental site is the LKAB dynamic support designed to be used in burst prone areas. It consists of 100 mm fibre reinforced shotcrete (40 kg/m³ steel fibre), 5.5 mm wire diameter welded steel mesh spaced on a grid of 75 mm x 75 mm, as well as 3.05 m long 20 mm diameter D-bolt or NMX-Dynamic bolt on a grid of 1 m x 1 m. The details of the instrumentation at the experimental site can be seen in Table 1. The layout of the instrumentation and instrumentation array are shown in Figure 1.

3.2.2 Monitoring results

Seven instrumented D-bolts, seventeen MPBXs and sixteen geophones were installed at level -1108, block 34 in September, 2015. Until now, the deformation of the rock mass and the instrumented bolts is small due to the short monitoring period. However, a seismic event with a local (IMS) magnitude of 1.5 occurred on September 29, 2015 near the experimental site and created small damage on the face of the nearby excavation and sidewall at the footwall drift, see Figure 5. Plots showing the location of the seismic event, the instrumented site together with the nearby mine-wide geophones are provided in Figure 5. The MPBX at Profile 8 (Figure 1) shows a sudden displacement jump immediately after the event and all of the 6 anchors (located within 4.2 m distance) move upward with similar amplitude around 0.2 mm, see Figure 6 (a). It may be attributed to the combined effects of slip and opening of joints or fractures located between the anchors and the assembly head. The results from the instrumented D-bolt also support this interpretation as no noticeable change can be observed from the displacement sensors along the 3.05 m long D-bolt. Again, the occurred damage proves that the experimental site is located at right place where high possibility of rockburst could be expected during the project period.

Comparison of ground motion in the solid rock (at 9 m depth from surface) and near the excavation surface (at 0.5 - 0.6 m depth) shows site amplification of particle velocity in the fractured zone near excavation surface (see Figure 6 (b)). The reason why the particle velocity near the excavation surface is amplified has recently been investigated by Zhang et al. (2015) using numerical analysis. The wave frequency, fracture stiffness, fracture spacing, and number of fractures (thickness of fractured zone) are the main factors that affected the velocity amplification according to the investigation.

Figure 5 Location of the seismic event and the damage and the location of the instrumented area, together with the nearby mine-wide geophones (a) horizontal view, and (b) vertical view. The seismic event (yellow sphere), rockburst damage (black ellipses), uniaxial geophones (blue rectangles) and tri-axial geophones (red triangles) are illustrated.
Figure 6 (a) Monitoring results from the MPBX installed in the roof at Profile 8, and (b) site amplification of particle velocity during the seismic event recorded in the sidewall at Profile 5.

3.3 Garpenberg Mine

3.3.1 Site description

The Garpenberg underground mine, around 180 km northwest of Stockholm, presumably one of the oldest mines still in production, having started mining in 1200. The mine produces complex ores containing zinc, lead, silver, copper, and gold. Boliden owns and operates the Garpenberg mine since 1957.

One of the major ore bodies in the Garpenberg mine is Lappberget, where the experimental site is located. The ore body is about 300 m long along the strike and from 15 to 100 m wide. The mining method used at the site is transverse sublevel open stoping with paste backfilling. Currently transverse stoping is applied by following a primary secondary approach in Lappberget, with stopes 20 to 30 m high and width from 10 to 15 m for primary and secondary stopes respectively. The experimental site is located at the level of 728 m in the sill pillar and the instruments are installed along the footwall drift.

The support system at the experimental site consists of 30 mm fibre reinforced shotcrete (30 kg/m$^3$ steel fibre), 5.5 mm wire diameter welded steel mesh spaced on a grid of 75 mm x 75 mm as well as 2.7 m long 25.4 mm diameter resin grouted rebar on a grid of 1.5 m x 1.5 m. 1.8 m long 22 mm diameter D-bolts are used in each mesh corner and in the mesh centre. The details of the instrumentation at the experimental site can be seen in Table 1.

3.3.2 Monitoring results

Fifteen instrumented fully grouted rebar rock bolts, twenty MPBXs and sixteen geophones were installed along the footwall drift in the sill pillar in August, 2015. The instrumented bolts were installed following the standard operational procedures and the displacement sensors for the instrumented bolts were arranged in a staggered configuration (see Figure 2). During the period of the investigation, mining (production blasting) was conducted in a stope near the experimental site on September 28 and October 10, 2015 (see Figure 7) and the ore was mined from 698 m progressively downward to 728 m below surface.

Temporal plots for one instrumented bolt and one MPBX installed in the drift roof of Profile 3 (P3 in Figure 7) are shown in Figure 8. Anchors No. 1 through 6 in Figure 8 (a) indicate the anchor positions for an individual MPBX, with Anchor No.1 closest to and Anchor No.6 farthest from the excavation. Positive values indicate that anchor and its corresponding assembly head moved away from each other while negative values mean that the two move towards each other. Unfortunately, the data-loggers could not be used
until September 28, 2015 and hence there were only manual readings from August 26 to September 28, 2015.

After the bolts were installed, the typical loads were in the range of 5 - 14 kN initially and then increased gradually. However, it was noted that several small jumps in the bolt load occurred immediately after production blasting. The same phenomenon was observed in the MPBXs installed around 1 m away from the instrumented bolt in the drift roof and large deformation occurred near excavation surface (Anchor 6 and Anchor 5 of the MPBXs). These events were synchronously detected by several bolts and MPBXs suggesting that they were related to definite changes in stresses and rock mass condition due to mining. Similar results have been observed by Hsiung and Ghosh (2006). General observation of the instrumented bolts and MPBX measurements and results indicated that the movements of the anchors were of two types. The first type is caused by stress re-distributions in the rock mass caused by mining and represented by a gradual increase in displacement. The second type of displacement exhibited a distinct pattern of step increase or decrease may be attributed to the combined effects of slip and opening of joints or fractures located between an anchor and the assembly head (Hsiung and Ghosh, 2006).

The displacement distributions in the rock mass and the distributions of axial strain in the bolts are plotted in Figure 9 (a) and (b), respectively. It can be seen that large amount of deformation occurred near the excavation surface within 1.3 m distance. The load (proportional to the strain) along the bolt is developed along the central section of the bolt at 0.8 – 2.1 m distance from the surface with the maximum load located at 1.1 m away from the surface. So far, the maximum strain along the bolt is still lower than the elastic limit (2500 microstrains) of the bolt during the monitoring period.

Figure 7 Layout of instrumented profiles (marked as P1, P2, ..., P7) and stopes blasted before and after the installation at the experimental site at the Garpenberg underground mine.
4 Conclusion

The ERSP project aims at developing new methods for evaluating the rock support performance in-situ that use all available information about i) the source of the seismic event (obtained from the local seismic network in the mine) and ii) the consequences of the seismic loading in terms of damage to the underground excavations and the rock support. Until now, four sites in three different mines have been instrumented.

Two damaging seismic events have occurred near the experimental sites at the Zinkgruvan Mine and the Kiirunavaara Mine after the instruments were installed. Even though they did not produce large disturbance and damage to the rock mass and rock support at the experimental sites, the results prove that the location selection of the experimental sites is right. The effects due to mining/production blasting have been monitored by the installed instruments which manifests the installation of the instruments is successful. The accumulative damage effect in rock and rock support has been captured by the instruments, which gives extremely important information for us to evaluate the loading and deformation.
history and further the excavation vulnerability potential before a seismic event occurs. The project is ongoing and more promising results are expected and could be obtained in this project.

**Acknowledgement**

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**References**


Experimental and numerical analysis of face pressure in EPB shield in East-West lot of line 7, Tehran subway

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Abstract
Face stability is one of the most important factors in selecting the tunnel excavation methods. This is particularly true for mechanized shield tunnelling, which has been developed for excavation in the soft soil and hydro-geological conditions. Most widely used mechanized shields, including earth pressure balance shields (EPBS) slurry shields and combined shields, which the collapse (active failure) and the blowout (passive failure) are controlled by the tunnel face support pressure in these shields. Therefore, to prevent any damages to surface and subsurface structures, determination of the appropriate amount of support pressure is necessary. Analytical, experimental and numerical methods are frequently used for the stability analyses of the tunnel face. In this article, the tunnel face stability of the east-west lot of Tehran subway line 7 has been investigated by analytical-experimental and numerical methods. Then, the obtained pressure values compared with real applied face pressure at 2000 meters of tunnel and the appropriate range of pressure has been selected for tunnel sections. In the following, with numerical modelling of this selective pressure range, the effects of this pressure range on the tunnel face stability have been investigated and the subsidence values which were caused by its application compared with actual values in the ground surface. By changing the soil mechanical parameters, water table condition and shield geometric parameters in the numerical model, the impacts of these parameters on the face pressure have been investigated. Finally, according to the geological and geotechnical conditions of the area, the most important parameters have been proposed.

Keywords: Face pressure, EPBS, Tunnelling, Tehran subway

1 Introduction
To excavate the underground spaces in urban areas, the Earth Pressure Balance Shield has been developed. Tunnel excavations under urban areas which often have soft soil conditions require special arrangement. To ensure the stability of the tunnel, surface structures and other underground structures are necessary that must be considered. Face pressure on the earth pressure balance shield is the most important factor to control the collapse (active failure) and blow-out surface (passive failure). The amount of subsurface subsidence and surface heave depends on the amount of face pressure (Gugliemetti et al. 2007). Calculation of the amount of face pressure is essential for extreme sensitivity, safe surface and underground structures. In this paper, face pressures were calculated by analytical experimental and numerical methods and then compared to the actual face pressure applied in the EPB shield.
2 PROJECT BACKGROUND

Tehran metro line 7 has two directions: east-west and north-south. East-west direction starts from Shahrak-e-Amir-al-Momenin in east of Tehran and continues under Mahalati highway, Moulavi St and Mohamadiyeh Square, the route changes at Navvab Safavi highway (North- South direction). The Length of east-west tunnel is 12050 meters that ends on N7 Shaft. This direction has excavated with EPB -TBM. Specification of this machine has been illustrated in table 1 (Sepasad Group). Figure 1 shows the main route of Tehran Metro Line 7.

Table 1 TBM specific data (Sepasad Group)

<table>
<thead>
<tr>
<th>TBM type</th>
<th>EPB</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cutterhead diameter (Bore Diameter)</td>
<td>9.164 m</td>
</tr>
<tr>
<td>Shield diameter</td>
<td>9.124 m</td>
</tr>
<tr>
<td>Length of Shield</td>
<td>10 m</td>
</tr>
<tr>
<td>Length of backup system</td>
<td>95 m</td>
</tr>
<tr>
<td>Cutting head Power</td>
<td>2100 w</td>
</tr>
<tr>
<td>Thrust pressure</td>
<td>65000 KN</td>
</tr>
<tr>
<td>Cutting head Speed</td>
<td>0-20 rpm</td>
</tr>
<tr>
<td>Number of segment</td>
<td>6+1+1</td>
</tr>
<tr>
<td>Rate of advance (Average for excavation 1.5 m)</td>
<td>60 min</td>
</tr>
</tbody>
</table>

This type of TBM is selected according to the geological and geotechnical conditions of the soil in the tunnel direction. The tunnel is divided into 6 geological units. Table 2 shows the values of the geotechnical parameters.

About 65 percent of the tunnel section is under the water table, which is most of the central and eastern tunnel path included. The tunnel does not confluence with known faults, but several important and active faults, such as Qasre Firozeh and North Rey are at distances from the tunnel (Sahel Consultant Engineers. 2010).

To calculate face pressure in different sections of the tunnel, according to the soil type, geology units encountered in the tunnel, the water level at the top of the tunnel and the thickness of the overburden, the tunnel was divided into 6 sections and face pressure calculated for them. Table 3 shows the sections of the tunnel.
Figure 1  Main route of Tehran Metro Line 7 (Sahel Consultant Engineers. 2010)

Table 2  Values of geology and geotechnical parameters proposed for geological units (Sahel Consultant Engineers. 2010)

<table>
<thead>
<tr>
<th>Units of engineering geology</th>
<th>Specific dry unit weight (γ) (kN/m²)</th>
<th>Poisson’s ratio (ν)</th>
<th>Modulus of deformation (E) (Mpa)</th>
<th>internal friction angle (φ) (degree)</th>
<th>Cohesion (KPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>ET-1</td>
<td>18.6</td>
<td>0.30</td>
<td>80</td>
<td>34</td>
<td>14</td>
</tr>
<tr>
<td>ET-2</td>
<td>18.4</td>
<td>0.30</td>
<td>75</td>
<td>33</td>
<td>15</td>
</tr>
<tr>
<td>ET-3</td>
<td>19.0</td>
<td>0.32</td>
<td>50</td>
<td>33</td>
<td>30</td>
</tr>
<tr>
<td>ET-4</td>
<td>18.2</td>
<td>0.30</td>
<td>50</td>
<td>32</td>
<td>22</td>
</tr>
<tr>
<td>ET-5</td>
<td>17.0</td>
<td>0.35</td>
<td>35</td>
<td>28</td>
<td>31</td>
</tr>
<tr>
<td>ET-6</td>
<td>17.0</td>
<td>0.35</td>
<td>10</td>
<td>27</td>
<td>0</td>
</tr>
</tbody>
</table>

3  Calculating the face pressure, experimental-analytical method

The minimum pressure for supporting the tunnel surface has been calculated by using various analytical and experimental methods. According to Table 2, which shows the values of geotechnical properties of soil, these values are very close. With regard to this issue, it can be assumed that the soil along the tunnel route is homogeneous. Excavation pressure values obtained from different analytical and experimental methods for all the sections of the tunnel are given in Table 4.
Table 3  Sections for calculated face pressure (Maboudi 2012)

<table>
<thead>
<tr>
<th>Number of section</th>
<th>Soil unit</th>
<th>Soil type (USCS)</th>
<th>Water level (m)</th>
<th>Thickness of the overburden (m)</th>
<th>Length of section (m)</th>
<th>special condition</th>
</tr>
</thead>
<tbody>
<tr>
<td>1A</td>
<td>ET2</td>
<td>SC, SC- SM, GC</td>
<td>0</td>
<td>12</td>
<td>6325</td>
<td>1A Z=12, h=0</td>
</tr>
<tr>
<td>1B</td>
<td>ET2</td>
<td>SC, SC- SM, GC</td>
<td>7</td>
<td>20</td>
<td>6325</td>
<td>1B Z=20, h=7</td>
</tr>
<tr>
<td>2</td>
<td>ET4</td>
<td>SC, SM</td>
<td>9</td>
<td>20</td>
<td>1210</td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>ET1</td>
<td>GW, GW- GM, SW, SP</td>
<td>13</td>
<td>20</td>
<td>980</td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>ET1,2</td>
<td>SC, GC, GW, SW, SP</td>
<td>13</td>
<td>20</td>
<td>1335</td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>ET2,4</td>
<td>SC, GC, SC, SM</td>
<td>11</td>
<td>20</td>
<td>1110</td>
<td></td>
</tr>
<tr>
<td>6A</td>
<td>ET2,3</td>
<td>SC, GC, GW, GW- GM, SW, SP</td>
<td>0</td>
<td>10</td>
<td>1090</td>
<td>6A h=0, z=10</td>
</tr>
<tr>
<td>6B</td>
<td>ET2,3</td>
<td>SC, GC, GW, GW- GM, SW, SP</td>
<td>10</td>
<td>20</td>
<td>1090</td>
<td>6B h=10, z=20</td>
</tr>
</tbody>
</table>

4  Numerical Methods Modelling

To determine the proper range of excavation pressure calculated by analytical-experimental methods it also should be modeled in numerically till the effects of this pressure on the structure could be seen. Excavating face stability is surveyed by numerical modeling. Two important factors in choosing the excavating pressure are ground subsidence due to the surface of the pressure applied by excavation machine and stability of tunnel face that these factors can be considered for numerical modeling. Numerical modeling can be done by different software such as: Plaxis 3D tunnel, Abaqus, Flac 3D. In this paper the modeling performed using Plaxis 3D. This software has been chosen for face stability analysis and modeling of the balance pressure shield of earth and it is easy to use and has good graphical interface for modeling.

Table 4  Excavation face pressure from analytical and experimental methods in (kPa)

<table>
<thead>
<tr>
<th>Sections</th>
<th>1A</th>
<th>1B</th>
<th>2</th>
<th>3</th>
<th>4</th>
<th>5</th>
<th>6A</th>
<th>6B</th>
</tr>
</thead>
<tbody>
<tr>
<td>Terzaghi</td>
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<td>200</td>
<td>245</td>
<td>249</td>
<td>220</td>
<td>73</td>
<td>216</td>
</tr>
<tr>
<td>Laca &amp; Domio</td>
<td>20</td>
<td>89</td>
<td>109</td>
<td>149</td>
<td>149</td>
<td>129</td>
<td>20</td>
<td>119</td>
</tr>
<tr>
<td>John Sekz &amp; Steiner</td>
<td>39</td>
<td>54</td>
<td>113</td>
<td>60</td>
<td>60</td>
<td>113</td>
<td>38</td>
<td>135</td>
</tr>
<tr>
<td>Kovari &amp; Angonsta</td>
<td>24</td>
<td>74</td>
<td>89</td>
<td>136</td>
<td>135</td>
<td>109</td>
<td>24</td>
<td>103</td>
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<tr>
<td>Broere</td>
<td>39</td>
<td>54</td>
<td>113</td>
<td>60</td>
<td>60</td>
<td>113</td>
<td>38</td>
<td>135</td>
</tr>
<tr>
<td>Karranza &amp; Torres*</td>
<td>59</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>54</td>
<td>-</td>
</tr>
<tr>
<td>Kerus</td>
<td>50</td>
<td>117</td>
<td>120</td>
<td>180</td>
<td>181</td>
<td>140</td>
<td>48</td>
<td>147</td>
</tr>
</tbody>
</table>

*This method uses for the equilibrium condition for material undergoing failure above the crown of a shallow circular (cylindrical or spherical) cavity. (Repetoo at al. 2006)

In section 1A the tunnel excavation is planned at the level of 2000. At this point, with having the excavating pressure applied by the excavation machine and the subsidence on the surface of the ground, a numerical
model calibrated. It will be provided for accuracy of data with what occurred in reality. A numerical model by the software was used for comparison between predicted subsidence and the real subsidence obtained at ground level. The model is calibrated to 700 meters of the excavation that has been selected randomly. Finite element analysis by Plaxis 3D tunnel software includes: drawing geometry, applying boundary conditions, assigning material properties, meshing, and applying the initial conditions due to the pressure in the presence of water and effective initial pressure. Firstly the model geometry was drawn according to the characteristics of the soil units listed in Table 2. Excavation machines installed the segments with a thickness of 35cm behind the tunnel sidewall. Figure 2 shows the 3D mesh model which is calibrated for the numerical model in the 700 meters tunnel excavation is intended. By applying 20 KPa traffic load in fixed situation, the model is ready for simulation.

![Figure 2 3D view of the calibrated model](image)

### 4.1 Outcome results

Figure (3a) shows the end displacement of the first computing phase. The total amount of displacement of the first phase is equal to 10.48 mm. In the second phase while the excavation machine is moving forward and installing the segment, the total amount of displacement is 13.28 mm. Figure (3b) shows the total displacement of the second phase. Figure 5 shows the total displacement of the calibrated model without pressure applied by the device. Total displacement of the model is 32.68 mm. With no pressure; the soil will move into the tunnel face. The uncontrolled entry of soil into the tunnel will damage the tunnel and surface structures.
Experimental and numerical analysis of face pressure in EPB shield in East-West lot of line 7, Tehran subway
V. Maboudi, F. Molaei, H. Siavoshi, and S. Rahimi

Figure 3  a) The displacement of the end of the first computing phase  b) the total displacement of the second phase

Figure 4  Total displacement of the calibrated model without applied pressure

By comparing the predicted subsidence in the calibrated numerical models in Table 6 and the real subsidence on ground level it can be seen that the values are close and so it can be concluded that the numerical model can be used to select the appropriate range of excavating pressure. Three points are selected in numerical models for drawing the load-displacement curve. These are: tunnel center, ground level in tunnel axis and ground level after segment installation. The chart is drawn on the basis of load-displacement factor. In this diagram, the load factor is equal to release rate of tension during the construction stage. Figure 5 shows the load factor of the total displacement. The total subsidence after exerting the pressure of 61 kPa on the surface after segments installation and the tunnel axis, respectively are 3.90 mm and 1.16 mm. Soil displacement of the tunnel center is equal to 3.45 mm. The subsidence was measured by an instrument in calibrated range is given in Table 6.
Surveying the pressure of the first excavation of 2000 meters of tunnel by boring machines shows that the excavating pressure is in the range of 48 to 80 kPa. This part of the tunnel is located in section 1A. Pressures calculated by the method of Atkinson and Potts, Karranza and Torres, Krause and Terzaghi are in the range of applied pressure by excavation machine (Repoto et al. 2006). As a result, the pressure measured in section 1A from those methods is close to the values obtained in numerical modeling. Due to the similarity of soil geomechanical parameters in tunnel route, we can apply these methods to other sections to calculate the pressures by numerical modeling. In section 1A and 6A the excavation is done without the presence of ground water. Atkinson & Potts, Karranza & Torres methods only apply for dry conditions, in which case the Krause method is considered for the pressure calculation (Broere, 2001). Hence, the pressure in Krause method was based on other sections and the pressure close to that were in the range of the excavating pressure. For example, in Section 5, the pressure obtained was 140 kPa in Krause method and that in Kovari & Angonsta, Leca & Dormieux, Jancsecz & Steiner, and Broere methods respectively 109, 129, 113, 113 KPa, which can be considered in numerical modeling. In Section 1B, the pressure within and close to Krause method has not been obtained. For this section Terzaghi & Krause pressure was applied in numerical models. In Table 7 the close pressures are brought together to survey and apply in numerical model and with exerting these pressures in the numerical model, the amount of the ground subsidence and its impact on the stability of the tunnel have been studied.
Figure 5  Load factor of total displacement for calibrated model in 61 kPa pressure

Table 7  Calculated pressures to apply in numerical model

<table>
<thead>
<tr>
<th>Section</th>
<th>Methods &amp; the amount of pressure in kPa</th>
</tr>
</thead>
<tbody>
<tr>
<td>1A</td>
<td>Krause, 50</td>
</tr>
<tr>
<td>6A</td>
<td>Krause, 48</td>
</tr>
<tr>
<td>2</td>
<td>Krause, 120</td>
</tr>
<tr>
<td></td>
<td>Krause, 140</td>
</tr>
<tr>
<td>6B</td>
<td>Krause, 147</td>
</tr>
<tr>
<td>3</td>
<td>Krause, 164</td>
</tr>
<tr>
<td>4</td>
<td>Krause, 177</td>
</tr>
<tr>
<td>1B</td>
<td>Krause, 117</td>
</tr>
</tbody>
</table>

5.2  Considering amount of the subsidence

In Section 1A, the highest subsidence on the ground surface (along the tunnel axis) is 2.8 mm, which is achieved from excavating pressure of 50 & 59 KPa. The amount of soil displacement in front of the
excavation machine at its maximum pressure of 50 kPa is estimated as 4.5 mm. This amount reflects a desire for the soil to enter tunnel. If the excavating pressure decreases this amount will increase. As a result, the excavating pressure in this section is recommended between 50 to 80 KPa that the exact amount of pressure applied by the excavation machine in the first 2000 meters of the tunnel. Thus, for this section the proper excavating pressure has been obtained from the methods: Atkinson & Potts, Karanza & Torres, and Krause & Terzaghi. In section 6A the maximum amount of subsidence in the tunnel at ground level was 2.5 mm by the pressure of 48 and 54 KPa. The maximum amount of soil displacement in the center of the tunnel and the excavation face is 1.7 mm. This is the result of the pressure of 48 KPa and is considered as the minimum excavating pressure. For this section, excavating pressure between 48 and 73 KPa is recommended. To calculate the excavating pressure for this section, the methods by Atkinson & Potts, Carranza and Torres Krause Terzaghi can be used. The Maximum amount of subsidence for section 2 in the tunnel axis and on the ground surface is 3.5 mm respectively is obtained at a pressure of 109, 120, 113 KPa. With the pressure of 120 KPa soil displacement in the front shield is 3.4 mm. Excavating pressure for this section is proposed between 100 and 120 KPa. To calculate the excavating pressure in section 2, the methods by Leca and Dormieux, Jancsecz and Steiner, Crus and Brewer approximate the propel pressure.

In section 5 the maximum subsidence on the ground and along the tunnel axis is 3.7 mm that has been achieved with the pressure of 109, 113, 129 KPa. Most soil displacement in the tunnel face by exerting a pressure of 140 KPa is 6.1 mm. In this section, with the increase of face pressure, the amount of displacement also increased. This increase represents soil displacement toward the tunnel face. Displacement- load factor curve is not able to express it, but at the end of the displacement computing phase, it can be obtained obviously. In this section excavating pressure was proposed between 109 to 140 KPa and the methods: Kovari & Angonsta, Leca & Dormieux, Broere, Jancsecz & Steiner Krause approximated the correct pressure. In section 6B the maximum amount of subsidence on the ground (the tunnel axis) is 3.4 mm with exerting pressure of 103 and 119 KPa has been achieved. Maximum soil displacement of 4.7 mm is in front of shield, which is achieved by exerting the pressure of 147 KPa. In this section, with increasing of excavation face, soil will move toward in front of the face. For excavating in this section, the pressure will be recommended between 103 to 147 KPa. In this section, the methods: Kovari & Angonsta, Leca & Dormieux, Broere, Jancsecz & Steiner Krause approximated the correct pressure.

In section 3 the maximum amount of subsidence on the ground (the tunnel axis) is 3.3 mm with exerting pressure of 136 and 149 KPa has been achieved. Maximum soil displacement of 8.6 mm is in front of shield, which is achieved by exerting the pressure of 164 KPa that shows the desire of soil movement toward the tunnel face. For excavating in this section, the pressure will be recommended between 136 to 164 KPa. In this section, the methods: Kovari & Angonsta, Leca & Dormieux, Broere, Krause approximate the correct pressure.

In section 4 the maximum amount of subsidence on the ground (the tunnel axis) is 3.7 mm with exerting pressure of 135 and 149 KPa has been achieved. For excavating in this section, the recommended pressure is between 135 to 177 KPa and then the maximum displacement will achieve (0.80 cm). In this section, the methods: Kovari & Angonsta, Leca & Dormieux, Broere, Krause approximated the correct pressure.

In section 1B the maximum amount of subsidence on the ground (the tunnel axis) is 3.5 mm by exerting pressure of 136 and 117 KPa has been achieved. Maximum soil displacement of 9.0 mm is in front of shield, which is achieved by exerting the pressure of 164 KPa. For excavating in this section, the pressure will be recommended between 117 to 147 KPa. In this section, the Krause method approximates the correct pressure.
6 Parametric studies and sensitivity analysis

To examine the factors influencing the face pressure, calibrated model is used. Mechanical properties of soil including adhesion and internal friction angle with respect to the range specified in the testing and geomechanical studies on the tunnel route are changed. Groundwater conditions and the depth of the tunnel need to be examined for sensitivity analysis. Soil mechanical parameters for the tunnel route are given in Table 8. The calibrated model parameters are kept constant and only the internal friction angle are varied. By choosing the lowest internal friction angle for soil units in numerical models, the ground subsidence in the tunnel axis is 3.9 mm and by exerting of the highest amount 8.7 mm. With increasing soil adhesion, the ground subsidence in the tunnel axis is 8.5 mm and for its decreasing it would be 4.0 mm.

Table 8 The values of internal friction angle and adhesion of soil units in the tunnel route (Sahel Consultant Engineers. 2010)

<table>
<thead>
<tr>
<th>Soil units</th>
<th>internal friction angle (degree)</th>
<th>adhesion of soil (kPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Minimum</td>
<td>Maximum</td>
</tr>
<tr>
<td>ET1</td>
<td>33</td>
<td>35</td>
</tr>
<tr>
<td>ET2</td>
<td>32</td>
<td>34</td>
</tr>
<tr>
<td>ET3</td>
<td>28</td>
<td>38</td>
</tr>
<tr>
<td>ET4</td>
<td>31</td>
<td>36</td>
</tr>
<tr>
<td>ET5</td>
<td>26</td>
<td>31</td>
</tr>
</tbody>
</table>

In the calibrated numerical model, ground-level subsidence on the tunnel axis is equal to 1.2 mm. By changing the angle of internal friction angles in the defined range, the ground-level subsidence for minimum and maximum values of this parameter, are respectively 3.9 and 0.8 mm. Soil adhesion values in the defined range by the highest and lowest changes as a result of its actions, ground-level subsidence in the lowest and highest value of this parameter will be 0.4 and 0.9 mm. In the calibrated model, the overburden of tunnel is 12 meters while this amount for sensitivity analysis has been increased to 20 meters. Changing in the tunnel overburden, the amount of ground-level subsidence in the tunnel axis estimated of 4.3 mm. According to the results of the modeling which has done for the east-west line 7 Tehran subway tunnel, water content, the depth of tunnel, adherence of soil and internal friction angle, respectively have the highest impact on face pressure. The predicted values of ground-level subsidence in the tunnel axis after the installation segment will be increased due to the conical shape of shield earth pressure balance and after the advances of excavation machines, ground will have movement. The proposed pressure for tunnel excavation and the appropriate methods for the tunnel pressure calculation are shown in table 10.

Table 9 The values of subsidence on the ground level in tunnel axis for sensitivity analysis

<table>
<thead>
<tr>
<th>subsidence on the ground level in tunnel axis mm</th>
<th>subsidence in the ground level in tunnel axis mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>Tunnel below water level</td>
<td>Tunnel above water level</td>
</tr>
<tr>
<td>5.9</td>
<td>1.2</td>
</tr>
<tr>
<td>20 meters overburden</td>
<td>12 meters overburden</td>
</tr>
<tr>
<td>4.3</td>
<td>1.2</td>
</tr>
</tbody>
</table>
Table 10 The proposed pressure and the appropriate methods for the tunnel pressure calculation

<table>
<thead>
<tr>
<th>Sections</th>
<th>Proposed pressure for tunnel excavation (kPa)</th>
<th>Appropriate methods for calculating the pressure</th>
</tr>
</thead>
<tbody>
<tr>
<td>1A</td>
<td>50-80</td>
<td>Atkinson &amp; Potts, Karranza &amp; Torres, Krause, Terzaghi</td>
</tr>
<tr>
<td>1B</td>
<td>117-147</td>
<td>Krause</td>
</tr>
<tr>
<td>2</td>
<td>100-120</td>
<td>Krause, Leca &amp; Dormieux</td>
</tr>
<tr>
<td>3</td>
<td>136-164</td>
<td>Krause, Kovari &amp; Angonsta, Leca &amp; Dormieux</td>
</tr>
<tr>
<td>4</td>
<td>135-177</td>
<td>Krause, Kovari &amp; Angonsta, Leca &amp; Dormieux</td>
</tr>
<tr>
<td>5</td>
<td>109-140</td>
<td>Krause, Kovari &amp; Angonsta, Leca &amp; Dormieux</td>
</tr>
<tr>
<td>6A</td>
<td>48-73</td>
<td>Atkinson &amp; Potts, Karranza &amp; Torres, Krause, Terzaghi</td>
</tr>
<tr>
<td>6B</td>
<td>103-147</td>
<td>Krause, Kovari &amp; Angonsta, Leca &amp; Dormieux</td>
</tr>
</tbody>
</table>

7 Conclusion

For estimating the excavating face pressure, experimental and numerical analysis methods can be used. Because of the different analytical and experimental methods to estimate the excavating pressure and the dependence on different parameters, numerical methods for accurate estimation of the pressure is inevitable. First, with identifying and classifying geological conditions along the tunnel, excavating pressure is calculated by different methods. The first 2,000 meters of the tunnel, the pressure of excavating exerted in TBM were compared with values calculated by the analytical and experimental methods and for this length, a calibrated model in numerical methods were used to validate the accuracy of the output values of the numerical method, thus the predicted subsidence in numerical methods were used to validate the accuracy of the output values in real section of drilled area of the tunnel. The pressure from analytical and experimental methods for each unit is used as an input for Plaxis 3D and by examining the values of subsidence and the displacement of soil in the ground in front of the tunnel center, the ground surface in tunnel axis and the ground surface after installation of segment, excavating pressure range is suitable for the tunnel sections along the proposed route that includes the excavating route. Finally, with parametric studies and sensitivity analysis, the effect of mechanical parameters of soil, tunnel depth and water conditions were examined. By changing the values of cohesion and internal friction angle of the soil within the limits specified in the experimental stage measured the effect of these parameters to decrease or increase the pressure of excavating were determined. Also, by changing the depth of the tunnel and water conditions, these two parameters were evaluated for face pressure. For this section calibrated numerical models is used. According to the results, in soil unit of Tehran Metro line 7 east-west tunnel, water condition, depth, adherence to the soil and the internal friction angle are influenced on the face pressure that should be considered to estimate the pressure on the excavation face.

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Experimental and numerical analysis of face pressure in EPB shield in East-West lot of line 7, Tehran subway

V. Maboudi, F. Molaei, H. Siavoshi, and S. Rahimi


Sepasad Group. "Summary of Tehran metro line 7 project". Sahel Consultant Engineers, February 2010, "Engineering geology study of the tunnel".
Failure mechanism related to accelerating creep of rock triggered by dynamic disturbance

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Abstract

It is well accepted that the presence of the large accumulated elastic energy in the rockmass is a prerequisite for rockburst occurrence, however, the external disturbances, such as the dynamic disturbance excited by blasting may be regarded as one of the necessarily key factors to trigger rockburst around the tunnel at depth. In addition, the rockburst that is triggered by dynamic disturbance often occurs some time after blasting, which result in difficulties in predicting the occurrence time of rockbursts. Therefore, the time-dependent creep of rock is one key factor to be studied for rockburst prediction. The rock failure under a wide range of strain rates including those under quasi-static geo-stress, creep and dynamic loading condition, should be investigated in order to investigate the full mechanism of time-delayed rockburst that is triggered by dynamic disturbance. Firstly, we performed multiple-stress-stepping creep tests on green sandstone under uniaxial stress condition. Secondly, a general-purpose damage-based model is developed to represent the damage of rock under creep and dynamic loading, based on which, a numerical program is developed. And the model was validated by comparing the numerical simulations with the experimental results of green sandstone. Thirdly, the numerical simulation on the accelerating creep of rock triggered by dynamic disturbance is conducted, in which the effect of creep stage and waveform of dynamic disturbance on damage and failure of rock is examined. This study may provide theoretical basis for mechanism of mining-induced rockburst triggered by dynamic disturbance.

1 Introduction

Rockbursts or “bumps” in underground mines are characterized by the spontaneous release of elastic energy, a portion of is transformed into kinetic energy, thus resulting in abrupt displacements and unpredictable rock failures (Müller, 1991). Rockbursts may cause severe devastation and endanger operation in mine. With an increase of mining depth, more and more cases of rockbursts have been reported (Ortlepp, 1994). It is well accepted that the occurrence of rockbursts is dependent on many factors such as in-situ stress conditions, geological structures, rockmass strength, underground excavation methods and the size of excavations (Durrheim, 1996; Mansurov, 2001; Mendecki, 1996; Wang, 2001), however, the external disturbances, such as the dynamic disturbance excited by rock blasting may be one of the necessarily key factors to trigger rockburst around the tunnel at depth. In addition, mining engineering practice indicates that rockburst occurs immediately or for a period after the blasting. Time-dependent behaviour of rock is important as it may cast light on why rockbursts do not always occur at blasting time but also when there is no external influence which could account for changes in stress distribution (Malan, et al. 1997). In this regard, the time-delayed rockburst is considered to occur when the rock mass is under creep that is firstly induced by the geo-stress and excavation, and then secondly triggered by a dynamic stress wave. Therefore, in order to reveal the mechanism of dynamic disturbances induced rock creep damage and triggered rockburst, we need to analyse the response of the rock mass under combined rheological and dynamic conditions.
Rock rheological damage and rock dynamics were usually studied separately, therefore, the effect of dynamic disturbances on rheological process of rock has not been considered yet. Moreover, numerical simulation on the creep damage effect of rock under dynamic disturbance is also lacking. Therefore, in this paper, firstly, we report results from multistage uniaxial creep experiments on green sandstone. Then, based on damage mechanics, we propose a damage-based constitutive law to simulate the creep behaviour of rock when the rock microstructure governed by different mineral grains and microdefects, is represented by rock heterogeneity. The damage-based model was validated against the experimental observations. Finally, the numerical simulation on the rock creep failure triggered by dynamic disturbance was conducted using the proposed model, in which the effect of in-situ stress conditions and waveform of dynamic disturbance on damage and failure of rock is investigated, in order to investigate the possible precursory features for the rockburst.

2 Uniaxial compression creep test

Creep experiments were performed on green sandstone specimens retrieved from a quarry at Neijiang Municipality, China. The composition is 60% quartz, 20% feldspar, 15% cement and 5% mica. No distinct layering or lamination is observed in specimen. The P-wave velocity is 2.63 km s\(^{-1}\) for the samples. The composition and physical properties of green sandstone are summarized in Table 1. The experimental curve of stress-strain under uniaxial compression test is shown in Figure 1. All samples were cored from the same block of material to a diameter of 50±0.02 mm. They were precision-ground to 100 ± 0.02 mm in length, with a length: diameter ratio of 2:1.

<table>
<thead>
<tr>
<th>Green sandstone parameter</th>
<th>Test results</th>
</tr>
</thead>
<tbody>
<tr>
<td>Density (g cm(^{-3}))</td>
<td>2.17</td>
</tr>
<tr>
<td>P-wave Velocity (km s(^{-1}))</td>
<td>2.63</td>
</tr>
<tr>
<td>Young's Modulus (GPa)</td>
<td>8.4</td>
</tr>
<tr>
<td>Uniaxial Compressive Strength (MPa)</td>
<td>29.8</td>
</tr>
<tr>
<td>Tensile Strength (MPa)</td>
<td>2.4</td>
</tr>
</tbody>
</table>

![Figure 1](image.png)

**Figure 1** Experimental curves of stress-strain under uniaxial compression test

The unconfined multi-stage creep experiments were performed at room temperature in a rock rheology test machine. The load increase is achieved with dead-weight loading system which is amplified by a hydraulic system loading the rock specimens. The maximum compressive load is 400 kN.
The average UCS of the green sandstone specimens was 29.8 MPa. We adopted the methodology of stress-stepping in which multiple brittle creep experiments can be conducted on a single sample, as described in Heap et al., (2009a). The load was maintained at 45%, 55%, 65%, 75% and 85% of the UCS of rock. In multistage creep experiments, the load is increased to the higher levels at 12-hour intervals.

In this section, we present the results of the multistage unconfined creep tests. Three green sandstone specimens (GS-W2, GS-W3 and GS-W5) were selected. Experimental curves of axial strain against time under multistage unconfined creep tests are shown in Figure 2. In multistage creep experiments, the force values increased to the higher levels at 12-hour intervals. As soon as the axial stress is increased, the axial strain increased with a high initial rate. Shortly prior to failure, each specimen experienced an accelerating deformation. We treated this phase as accelerating creep. In the following analysis, emphases will be given to test GS-W2.

![Figure 2 Experimental curves of axial strain against time under multistage unconfined creep tests](image)

Figure 3 shows the curves of axial strain against time for the specimen GS-W2 under different stress levels. It is important to note that the five curves are obtained from the same sample (GS-W2), each curve corresponding to a different stress step. An offset was applied in order that the five curves have the same origin of time. Figure 3 clearly shows the specimen produce some amount of instantaneous strain in the loading process. The values of instantaneous response are dependent on the stress levels.

![Figure 3 Curves of axial strain against time curve for the specimen GS-W2 under different stress levels](image)

3 Damage-based constitutive law for rock creep

It is generally known that accelerating creep originates from microcrack interaction (Brouard et al., 2013; Molladavoodi and Mortazavi, 2011). Numerical simulation of creep processes must reflect the progressive
degradation of rock. In this regard, the elastic damage-based constitutive relationship is incorporated to describe the accelerating creep of rock.

3.1 Creep model of rock

The total strain consists of elastic ($\varepsilon_e$) and creep strain ($\varepsilon_c$), i.e.,

$$\varepsilon = \varepsilon_e + \varepsilon_c.$$  \hspace{1cm} (1)

The constitutive laws most often used to define the creep behavior of rock are based on a classical power law equation of the following type (Aubertin, et al., 1991; Yahyaa, et al., 2000; Nabarro, 2004; Su, et al., 2013). As for isothermal conditions, it reads

$$\dot{\varepsilon}_c = A \sigma^\alpha \beta^{\beta-1}. \hspace{1cm} (2)$$

It could also be extended for triaxial stress condition,

$$\dot{\varepsilon}_{ij} = \frac{3}{2} S_{ij} A \sigma_c^{\alpha-1} \beta^{\beta-1}$$ \hspace{1cm} (3)

where $\dot{\varepsilon}_{ij}$ is the creep strain rate, $S_{ij}$ is the deviatoric part of $\sigma_{ij}$, and $\sigma_c$ is effective stress defined as

$$\sigma_c = \left(\frac{1}{\sqrt{2}}\right) \sqrt{(\sigma_{11} - \sigma_{22})^2 + (\sigma_{33} - \sigma_{22})^2 + (\sigma_{11} - \sigma_{33})^2 + 6(\sigma_{12}^2 + \sigma_{23}^2 + \sigma_{13}^2)}.$$ \hspace{1cm} (4)

This creep model can describe the decelerating creep, but it fails to represent the accelerating creep. In the following context, a damage-based model for representing the accelerating creep of rock is proposed.

3.2 Damage-based constitutive model

The mechanical properties (the elastic modulus and uniaxial compressive strength, etc.) of rock are heterogeneous and assumed to conform to a two-parameter statistical Weibull distribution (Zhu and Tang, 2004). The elastic damage constitutive law of rock under uniaxial stress condition is illustrated in Figure 4 (Zhu and Tang, 2004). The stress-strain curve of each element is considered linear elastic until the given damage thresholds are attained. We choose the maximum tensile stress criterion and modified Mohr-Coulomb criterion as the damage thresholds to judge the tensile damage and shear damage of elements, respectively, which are expressed as:

$$F_1 \equiv -\sigma_3 - f_{c0} = 0 \quad \text{and} \quad F_2 \equiv \sigma_1 - \sigma_3 \frac{1 + \sin \phi}{1 - \sin \phi} - f_{t0} = 0$$ \hspace{1cm} (5)

where $\sigma_1$ and $\sigma_3$ are the major and minor principal stresses respectively, $f_{c0}$ is the uniaxial compressive strength and $\phi$ is the internal friction angle, $f_{t0}$ is the uniaxial tensile strength, $F_1$ and $F_2$ are two damage threshold functions. The tensile strain criterion is always used with priority to judge whether the element is damaged or not. If the element does not damage in tensile mode, the Mohr-Coulomb criterion is then used to judge whether the element damages in shear modes or not. The sign convention used throughout this paper is that compressive stresses are positive and tensile stresses are negative.
According to the elastic damage principle, the elastic modulus of an element degrades monotonically as damage evolves, and the elastic modulus of damaged material is expressed as:

$$E = (1 - D) E_0 \quad (6)$$

where $D$ represents the damage variable, which lies between 0 and 1, and $E$ and $E_0$ are the elastic moduli of the damaged and the undamaged material, respectively. According to the constitutive law as shown in Figure 4, the damage variable $D$ can be described as follows:

$$D = \begin{cases} 
0 & F_1 < 0 \text{ and } F_2 < 0 \\
1 - \frac{\varepsilon_{u0}}{\varepsilon_3} & F_1 = 0 \text{ and } dF_1 > 0 \\
1 - \frac{\varepsilon_{c0}}{\varepsilon_1} & F_2 = 0 \text{ and } dF_2 > 0 
\end{cases} \quad (7)$$

where $\varepsilon_1$ and $\varepsilon_3$ are major principal strain and minor principal stain, respectively. $\varepsilon_{u0}$ and $\varepsilon_{c0}$ are the maximum principal strain in tension and the maximum principal strain in compression when damage occurs according to the maximum tensile stress criterion and Mohr-Coulomb criterion, respectively, and $n$ is a constitutive coefficient with a value of 2.0.

In this respect, the damage variable calculated with Eq. (7) is always from 0 to 1.0 regardless of what kind of damage it may suffer. However, in the damage zone figure, in order to distinctly display the two kinds of damage modes (i.e. tensile damage and shear damage) in the post-processing figures, the tensile damage is represented as negative numbers, while the shear damage is represented as positive ones. When damaged or failure elements are clustered, it may lead to macroscopic failure, i.e. occurrence of accelerating creep of rock. Therefore, although the Eqns. (5)-(7) are time-independent, it can be capable of describing the time-dependent evolution of damage when it is coupled with the creep model as given in Eq. (3) to describe the creep behavior of rock (Xu and Tang, 2012).

### 3.3 Validation against experimental observations

To validate the feasibility of the model, a uniaxial brittle creep with different stress level steps was simulated to compare with the previous experiment data. According to the Weibull distribution, the mesoscale physic-mechanical properties of the individual elements are listed in Table 2. The creep parameters are listed in Table 3.
Table 2  Weibull distribution parameters of green sandstone

<table>
<thead>
<tr>
<th>Rock parameter</th>
<th>value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Homogeneity index</td>
<td>3.0</td>
</tr>
<tr>
<td>Mean young’s modulus/GPa</td>
<td>9.5</td>
</tr>
<tr>
<td>Mean uniaxial compressive strength/MPa</td>
<td>90</td>
</tr>
<tr>
<td>Frictional angle/°</td>
<td>30.2</td>
</tr>
<tr>
<td>Ratio of compression to tension strength</td>
<td>12.4</td>
</tr>
<tr>
<td>Passion ratio</td>
<td>0.29</td>
</tr>
</tbody>
</table>

Table 3 Creep parameters of green sandstone

<table>
<thead>
<tr>
<th>Creep parameter</th>
<th>value</th>
</tr>
</thead>
<tbody>
<tr>
<td>A/ s⁻¹</td>
<td>1.8 × 10⁻⁸</td>
</tr>
<tr>
<td>α</td>
<td>1.5</td>
</tr>
<tr>
<td>β</td>
<td>0.3</td>
</tr>
</tbody>
</table>

The geometry of the modelled sample was 50 mm×100 mm and it was discretized into a 100 × 200 (20000 elements) square grid (i.e. each square element had side length of 0.5 mm). In the simulation, there is five loading steps (13.5 MPa, 16.6 MPa, 19.5 MPa, 22.4 MPa and 25.3 MPa), every step lasts 12 h.

![Damage evolution](image)

**Figure 5  Damage evolution**

The damage distributions in different loading time are presented in Figure 5. At the first stress stage (t=6h), the initial damage distribute stochastically in the sample; At the second stress stage (t=18 h), there is more damages were seen to be distributed in the sample; At the third stress stage (t=30 h), the damage zones becomes more and more accumulated; At the fourth stress stage (t=42 h), with the increasing of the loading time, there is some small cracks due to the cluster of the damage; At the last stress stage, there is a through-going macro shear crack leading to the collapse of sample.
The simulated failure mode is plotted in Figure 6, together with the previous experimental one. Figure 6 shows that the simulated failure mode is in good agreement with the experimental failure mode.

![Figure 6](image_url)

**Figure 6** Comparison between numerical simulation result and experimental result

As shown in Figure 7, the numerically simulated creep curves are in good agreement with the experimental results. In this regard, the numerical simulations accurately capture the character of a classic experimental creep curve, that is, all the creep stages are observed. Based on these validations, the damage-based model is feasible and Reliable.

**Figure 7** Comparison between simulated and experimental creep curves

4. Numerical simulation on accelerating creep of rock triggered by dynamic disturbance

The numerical simulation on accelerating creep of rock triggered by dynamic loading includes two steps: the first step is to analyse the damage process of rock under multi-stage creep, and the second step is to implement the dynamic analysis of pre-stressed rock after it is triggered by dynamic disturbance. In the second step, the damage induced by initial creep is also included, thus the effect of dynamic disturbance on the creep damage of rock can be modelled.

4.1 Numerical model

The specific geometries and loading conditions for this model are shown in Figure 8. For the sake of simplicity, the analyses presented here are performed under plane strain condition. The rock is assumed to be heterogeneous with its mechanical properties defined according to a Weibull distribution. The rock has been studied is the green sandstone, with physico-mechanical parameters as listed in Table 2. The creep parameters are listed in Table 3.

The numerical simulations on creep tests were performed under three stresses (\( p_s = 0 \) MPa, 10 MPa and 15 MPa, respectively) and sustained for 12 h. The dynamic loading conditions, as shown in Figure 8, is a
trapezoid shape stress pulse $p_d(t)$, as defined in Figure 8b, is applied at the top boundary of model domain. Meanwhile, the constant static stress ($p_s$) still applied in the model.

![Diagram](image-url)

(a) Model setup

(b) Dynamic disturbance $p_d(t)$

Figure 8 (a) Geometries and loading conditions for rock specimen subjected to creep and dynamic loading. (b) Incident stress waves imposed on the top of rock specimen: (I) $p_{dm} = 20$ MPa; (II) $p_{dm} = 30$ MPa; (III) $p_{dm} = 40$ MPa.

4.2 Numerical results

4.2.2 The accelerating creep of rock triggered by dynamic disturbance

Figure 9 showed the damaged zone and Young’s modulus during creep of rock specimens triggered by dynamic disturbance with amplitude $p_{dm} = 30$ MPa and duration $t_{dm} = 10 \mu$s. Damage of elements causes degradation of their elastic modulus and the elements totally damaged in tensile mode are displayed as black. It is found that the incident stress wave travels downwards along the rock specimen. Beside the initial damage zone occurred by creep static stress, the damage of several elements is induced at $t=10 \mu$s. When the incident stress wave travels downwards, more cracks are initiated near vertical axial line of the specimen because the stress wave reflection at the left and right boundaries. Therefore two obvious cracking zones are initiated. Similar to the modelling results of rock specimen subjected to dynamic disturbance using the same software as that used in this paper (Zhu, 2008), the two spalling zones distributed parallelly along the specimen are generally caused by the lateral tensile stress and which was induced by the creep static stress and incident compressive stress wave in axial direction. At the last time step ($t=50\mu$s), the two spalling zones passes through the rock specimen and therefore the rock specimen is complete failure.

4.2.3 Effect of dynamic disturbance

As shown in Figure 10, when the amplitude is 30 MPa, there are two spalling zones with the incident stress wave propagates downwards. When the amplitude is 40 MPa, it is clearly seen that the whole model domain is damaged. Under the increasing amplitude of dynamic disturbance, more energy is input into the rock specimen, the cracking zones are more widely distributed, and the rock in the failure zone are much more fragmented, suggesting that more violent failure can be triggered.
Figure 9  The creep failure process of rock specimen triggered by dynamic disturbance (creep-II, and dynamic disturbance with $p_{dm} = 30$ MPa)

Figure 10  The damage zone during the creep failure process of rock specimen under three creep stages (creep-II) and dynamic disturbance with varied amplitudes ($p_{dm} = 20$ MPa, $p_{dm} = 30$ MPa and $p_{dm} = 40$ MPa).

5 Conclusion

The rock failure under a wide range of strain rates including those under quasi-static stress, creep and dynamic loading condition, should be investigated in order to clarify full mechanism of time-delayed rockburst that is triggered by dynamic disturbance. In the paper, uniaxial compression creep experiments were performed on green sandstone specimens to investigate its creep behavior under multiple stress levels. Then we presented a damage-based constitutive law to describe the accelerating creep of rock. Our
model was validated by reproducing the experimental observation and then was used to simulate the creep failure of rock triggered by dynamic loading. The following conclusions can be drawn from the experiments and the numerical simulations:

1. The classic decelerating and accelerating phases were observed in stress-stepping creep experiments. Creep strain rates are shown to be highly dependent on the stress level.

2. The numerical simulations effectively capture the characteristics of a classic experimental creep curve, i.e., decelerating and accelerating creeps. The damage-based creep model can be used to investigate the creep response of inhomogeneous rock.

3. The creep static stress and the magnitude of dynamic disturbance are two important factors controlling the failure of rock specimen. The dynamic disturbance may contribute to the failure of rock specimen only when its amplitude is high enough. The increase of the damage make dynamic disturbance prone to trigger the creep failure.

Acknowledgements

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References


Fibre reinforced spray concrete: minimum performance requirement to meet safety needs

B. De Rivaz, Bekaert Maccaveri Underground Solutions BVBA, Belgium

Abstract

Multiple research studies and tests on the behaviour of steel fibre reinforced concrete have been carried out in recent years in various countries. They have greatly contributed to a better characterization of Steel Fibre Reinforced Concrete (SFRC), and have thus allowed to gain a better understanding of the behaviour of this material and to specify minimum performance requirements for each project.

This article will present the material property determination using standardized testing methods and some improvement in the test procedure for sprayed concrete in order to:

- obtain a mechanical property to be used as input for the dimensioning method
- be in line with International recommendation as model code 2010, edited by FIB

European standard EN 14487-1 mentions the different ways of specifying the ductility of fibre reinforced sprayed concrete in terms of residual strength and energy absorption capacity. It also mentions that both ways are not exactly comparable.

The energy absorption value measured on a panel can be prescribed when - in case of rock bolting - emphasis is put on energy which has to be absorbed during the deformation of the rock. This is especially useful for primary sprayed concrete linings (EN 14488-5: Testing sprayed concrete, part 5: Determination of energy absorption capacity of fibre reinforced slab specimens).

The residual strength can be prescribed when the concrete characteristics are used in a structural design model.

The performance of steel fibre reinforced (SFRC) concrete can be tested in different ways. In this paper, two methods are described to evaluate the post-crack behaviour of SFRC.

1 EN14488-5 square panel test

1.1 Minimum requirement

The plate test EN 14488-5 is designed to determine the absorbed energy from the load / deformation curve as a measure of toughness. The test is designed to model more realistically the biaxial bending that can occur in some applications, particularly in rock support. The central point load can also be considered to replicate a rock bolt anchorage. This test has proved to be of considerable benefit.

The square panel test (see Figure 1), also called the EFNARC panel test, is simulating at a laboratory scale the structural behaviour of the system anchor bolt - sprayed concrete under flexural and shear load.
This test allows to check the load bearing capacity for an imposed deformation and to control the capacity of absorbing energy under large deflections. Different reinforcement systems can be tested and compared and performance criteria can be established for different tunnel types and ground conditions. The panel test is much more appropriate than the beam test to determine the performance of a SFR (steel fibre reinforced sprayed concrete):

- A panel corresponds much better with a real tunnel lining than a beam; the panel support on the 4 edges simulates the continuity of the sprayed concrete lining.
- As in reality, steel fibres act in at least two directions and not just in one direction, which is the case in a beam test; the fibre reinforcing effect in a panel is very much similar to the real behaviour of a SFRS lining.
- SFRS can be compared very easily with a mesh reinforced sprayed concrete to be tested in the same way.
- No numerical material properties, such as post-crack strength values, can be determined from the square panel test due to an irregular crack pattern; however this has never been the intention of this test method; this method serves to quantify and illustrate the ductile behaviour of a steel fibre reinforced sprayed concrete tunnel lining.

In this test, a fibre reinforced slab specimen is subjected to a load, under deflection control, through a rigid steel block positioned at the centre of the slab. The load-deflection curve is recorded and the test is continued until a deflection of at least 30 mm is achieved at the centre point of the slab. From this curve, a second curve is calculated, giving the absorbed energy as a function of the slab deflection.

The slab specimens need to be prepared according to the regulations of EN14488-1. A mould with inner dimensions 600 x 600 mm, and an inner height of 100 mm shall be positioned within 20° of the vertical (unless another orientation has been specified) and sprayed with the same equipment, operator, technique, layer thickness per pass and spraying distance as the actual work. Immediately after spraying the thickness of the concrete specimens shall be trimmed to a thickness of 100+5 mm.

In practise we should know, that we mainly spray directly in a panel 600*600*100 worldwide, which is the best and easiest procedure.

The test (see Figure 2) shall be displacement controlled, with a constant rate of 1-0.1+0.1 mm/min at the centre of the slab. The load and deflection shall be continuously recorded with the data logger of the XY-plotter until a deflection of at least 30 mm is obtained.
Figure 2  EN14488-5 Testing on steel support

The result that needs to be expressed is the energy absorption until a deflection of 25 mm is obtained, which can be calculated as the area under the load-deflection curve between 0 and 25 mm deflection. This procedure was established for steel fibre to compare the behaviour with steel mesh assuming a similar mode of failure.

The main performance criteria that can be applied for a reference concrete C30/37 are described in the EN 14 487. According the geological and geotechnical context. It must be determined for each project.

However, Asquapro guideline (Larive, 2013) provides the following indications:

Table 1  Specification for a classic concrete class C25/30 to C30/37 at 28 days

<table>
<thead>
<tr>
<th>Application</th>
<th>Minimum energy absorbing class</th>
<th>Energy absorption in J, for a 25 mm arrow</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sprayed concrete acting as a protective skin, and for tough rocks / soils</td>
<td>E500</td>
<td>500</td>
</tr>
<tr>
<td>Sprayed concrete acting as a resistant skin, and for medium rocks / soils</td>
<td>E700</td>
<td>700</td>
</tr>
</tbody>
</table>

Beyond a C30/37, the energy values must be higher and the fracture ductility of concrete verified. Thus, for a C40/50 concrete, Asquapro proposes the following requirements (table 2).

Table 2  Specification for a concrete class C40/50

<table>
<thead>
<tr>
<th>Application</th>
<th>Minimum energy absorbing class</th>
<th>Energy absorption in J, for a 25 mm arrow</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sprayed concrete acting as a protective skin, and for tough rocks / soils</td>
<td>E800</td>
<td>800</td>
</tr>
<tr>
<td>Sprayed concrete acting as a resistant skin, and for medium rocks / soils</td>
<td>E1000</td>
<td>1000</td>
</tr>
</tbody>
</table>

The Barton’s Chart (see Figure 3) introduces performance criteria for SFRC based on EN plate test only:
A central point load is imposed on a round specimen measuring 75 mm thick x 800 mm diameter, supported on three radial points located on a 750 mm diameter. The use of three pivoted supports ensures that load distribution is always defined.

- There is no multi-crack, as the EN plate test (continuous support) does not allow any load redistribution.
- There is no simulation at a laboratory scale of the structural behaviour of the system anchor bolt - sprayed concrete under flexural and shear load, as is the case with the EN plate test or Norway round plate test.
- There is no correlation in terms of energy absorption between the EN plate test and the RDP test as we have between the Norway round plate test and EN plate test. Both are realized on continuous support. We cannot compare a test with continuous support (Under terminal panel) with a test acting as a beam test (Round Panel Determinal). We cannot compare different deflections at 25 mm Square Panel and 40 mm, even 80 mm RDP test.
- The central deflection up to 5 mm is applicable to situations in which the material is required to hold cracks tightly closed at low levels of deformation. Examples include final linings in underground civil structures such as railway tunnels that may be required to remain water-tight. The energy absorbed up to 40 mm is more applicable to situations in that the material is expected to suffer severe deformation in situ (for example, shotcrete linings in mine tunnels and temporary linings in swelling ground). Energy absorption up to intermediate values of central deflection can be specified in situations requiring performance at intermediate levels of deformation. A displacement of 40 mm is used to assess performance at high levels of deformation where large crack can be tolerated.
- 40 mm deflection on RDP test mean 10mm crack opening (Model code 2010 refer at 0,5 mm for SLS and 2,5 mm for ULS).
2 Restrictive specifications for reinforced sprayed concrete for underground support

The minimum recommendation for sprayed concrete is an absorbed energy value greater than 500J. The following graph (Figure 4) shows that such a value can be obtained when the concrete matrix is of good quality, but it does not ensure good post-crack stress absorption (sharp drop and limited post-peak absorption).

![Graph showing load-deflection curve with energy absorption]

**Figure 4** Unsatisfactory load-deflection curve in spite of 500J energy

However, a higher energy absorption value does not necessarily guarantee the appropriate behaviour for the substrate (cf. Figure 1 which represents a concrete that exceeds 800J).

Consequently, Asquapro proposes to analyse each of the curves obtained according to the test in EN 14488-5 as follows (at least three curves per test):

1. The maximum load of the elastic zone ($F_{el-max}$) must correspond to a deflection value less than 2 mm.
2. The minimum load after cracking and up to a deflection equal to 5 mm must be greater than 70% of $F_{el-max}$.

Based on the study of a significant number of curves (conducted by test laboratory Sigma Béton), this 70% value seems appropriate to select quality concretes. It allows, for example, the concrete in Figure 4 to be rejected, in spite of its 500J energy.

![Graph showing typical energy absorption curves]

**Figure 5** Typical energy absorption curves
Furthermore, Asquapro also proposes to specify the following points (some of these are included in the requirements of the standard but are not always observed in practice):

- Prepare 4 slabs for the energy absorption capacity test (3 + 1 back-up) to obtain average values over at least 3 test runs.
- Strictly observe the thickness of the slabs: 100 mm, +5 mm, -0. If their thickness exceeds 105 mm, the slabs are rejected.
- The slabs must still be whole after the test.
- In addition to the customary requirements, the test reports must include photos of the interior sides of each slab after testing, possibly after water spraying to clearly reveal the multicracking phenomenon.

Criterion for conformity: three tested slabs should not exhibit any value less than the specified energy.

3. **EFNARC THREE POINT BENDING TEST ON SQUARE PANELS WITH NOTCH**

3.1 **Test method description**

A practical method to determine the tensile behaviour of SFRC for shotcrete applications is a 3-point bending test on square panels (see Figure 6). This test combines the output of the EN14651 with the advantages of the EN14488-5 test (the same moulds can be used and due to the larger cracked section, the scatter is lower).

Disadvantages are the weight of the specimens (EN14651 beams are more user friendly) and the attention that needs to be paid to finishing the sprayed surface in order to execute a perfect 3-point bending test. After all, the rollers need to be in contact with the concrete specimen over the whole length.

![Figure 6 Three point bending test on square panel with notch](image)

This test method is promoted by EFNARC for the following main reasons:

- The geometry and dimensions of the specimens, as well as the spray method adopted will ensure distribution of the fibres in the matrix, which is close as possible to that encountered in the real structure.
- The dimensions of the test specimen will be acceptable for handling within a laboratory (no excessive weights or dimensions).
• The test will be compatible, as far as the experimental means permit, with use in a large number of standard equipped laboratories (no unnecessary sophistication).

• The geometry will be the same as in the plate test for Energy Absorption

• The plate could be sprayed on the job site.

• No need to sawn a prism from a panel which influences the result. The notch will provide a slower cracking process, thereby reducing the risk of a sudden fall

• By analogy with EN 14651, this test defines the residual flexural strength \((fr1, fr3)\) according to the updated international standard (MODEL CODE 2010). The mechanical property obtained will serve as input for the dimensioning method.

The slab specimens need to be prepared according to the regulations of EN14488-1. A mould with inner dimensions 600 x 600 mm, and an inner thickness of 100 mm shall be positioned within 20° of the vertical (unless another orientation has been specified) and sprayed with the same equipment, operator, technique, layer thickness per pass and spraying distance as the actual work. Immediately after spraying, the thickness of the concrete specimens shall be trimmed to a 1000+5 mm. It is very important to make sure that the spraying side of the specimen is perfectly flat, otherwise problems can be caused during testing.

This requirement is certainly the point to evaluate with more experience from job site and see the best practise to implement in the future.

We should use very good formwork and smoothen the upper surface immediately after spraying. This is a key requirement in order to:

• get a perfect three point bending test, as the rollers should be in contact over the whole line with the specimens

• avoid problems in the beginning of the test to stabilize and end up with a perfect linear curve in the elastic part of the test (due to the roller/specimen contact, which is not constant)

• avoid problems to control the test after the first crack.

Supports are stiff in one direction and moving in another one.

The notches are made with a table saw. The flat surface, which is in contact with the mould is resting on the plate of the table saw. When the table saw is cutting in this way, the notch depth is not constant over the whole area, but this is not important. The section which is left, is constant in this way, and it is this value which is used during the calculations. To be even closer at the real value, we measure at the two sides, and take the average.

The testing machine should be capable of operating in a controlled manner, producing a constant rate of displacement (CMOD or deflection), and have a sufficient stiffness to avoid unstable zones in the load-CMOD curve or the load-deflection curve. A total stiffness of the system of 200 kN/mm (including frame, load cell, loading device and supports) is advised.

All rollers should be made of steel and have a circular cross section with a diameter of 30-1+1 mm. Two of the rollers, including the upper one, shall be capable of rotating freely around their axis and of being inclined in a plane perpendicular to the longitudinal axis of the test specimen. The distance between the centres of the supporting rollers shall be equal to 500-2+2 mm.

The load measuring device needs to have an accuracy op 0.1 kN and the linear displacement transducer an accuracy of 0.01 mm. The data recording system should be able to record load and displacement at a rate not less than 5 Hz.

In the case of a testing machine controlling the rate of increase of CMOD, the machine shall operate from the start of the test with a CMOD-increase of 0.05 mm/min and data logging at minimum 5 Hz.
CMOD = 0.19 mm, the machine shall operate at a CMOD-increase of 0.18 mm/min and a minimum data logging of 1 Hz. The test shall not be terminated before a CMOD value of 3.5 mm is obtained.

In case of controlling the increase of deflection, the machine shall start the test with a deflection increase of 0.06 mm/min with a data logging of minimum 5 Hz. When the deflection reaches 0.26 mm, the deflection increase shall be changed to 0.25 mm/min until a final deflection of 4.5 mm, and a data logging of minimum 1 Hz.

If the crack starts outside the notch, the test result should be rejected.

The test results (see Figure 7) which need to be expressed are the limit of proportionality (LOP) and the residual flexural strength.

The limit of proportionality $f_{ct,L}$ is calculated as:

$$f_{ct,L} = \frac{3}{2} \cdot F_L \cdot \frac{l}{bh^2}$$  \hspace{1cm} (1)

Where $F_L$ is the maximum load between CMOD 0 and 0.05 mm or deflection 0 and 0.08 mm.

The residual flexural strength $f_{R,x}$ needs to be evaluated at four different displacements.

$$f_{R,i} = \frac{3}{2} \cdot F_{R,i} \cdot \frac{l}{bh^2}$$  \hspace{1cm} (2)

Where $F_{R,i}$ is the residual load at:

- $i = 1$: CMOD = 0.46 mm or deflection 0.63 mm
- $i = 2$: CMOD = 1.38 mm or deflection 1.89 mm
- $i = 3$: CMOD = 2.30 mm or deflection 3.16 mm
- $i = 4$: CMOD = 3.22 mm or deflection 4.42 mm

$l$ = the span between the supports (nominal distance 500 mm)

$b$ = the width of the concrete sample (nominal value 150 mm)

$h$ = the residual height of the concrete sample (nominal value 125 mm)

![Figure 7](image_url) Load displacement curve of a 3-point bending test on square panels
The dimensions of the plates in a 3-point bending test on square panels are different than the dimensions of the beams in the EN14651 test. Because of this, the relation between the CMOD and the deflection is different as well.

Three definitions need to be taken into account (see also Figure 8):

- CMOD: crack mouth opening displacement: linear displacement measured at the bottom of the notch of the beam
- Deflection: linear displacement, measured by a transducer, between the bottom of the notch and the horizontal line which connects the points located in the middle of the beam, above the supports.
- CO: Crack opening: linear displacement measured at the top of the notch of the beam

![Figure 8 Definition of crack opening, CMOD and deflection](image)

The purpose is to evaluate the 3-point bending test on square panels at the same crack opening as the EN14651 beam test. The next formulas approach the geometrical correlation between CMOD, deflection and crack opening:

\[
\text{crack opening} = \frac{4 \cdot \text{deflection} \cdot (0.9 \cdot h)}{\text{span}}
\]

(3)

\[
\text{CMOD} = \frac{4 \cdot \text{deflection} \cdot (0.9 \cdot h + \text{notch depth})}{\text{span}}
\]

(4)

Where:

- \(\text{span}\) is the distance between the supports (nominal value 500 mm)
- \(h\) = the residual thickness of the concrete specimen (nominal 125 mm for the EN14651 beams and 90 mm for the square panels)
- notch depth is the depth of the saw cut (nominal 25 mm for the EN14651 beams and 10 mm for the square panels)

To evaluate the residual strength at the same crack openings as in EN14651, the loads need to be recorded at the CMODs or deflections which are mentioned in the table below (Table 3).

<table>
<thead>
<tr>
<th>What</th>
<th>EN14651 CMOD</th>
<th>3-point bending test on plates CMOD</th>
</tr>
</thead>
<tbody>
<tr>
<td>CMOD</td>
<td>Mm</td>
<td>Mm</td>
</tr>
<tr>
<td>Deflection</td>
<td>Mm</td>
<td>Mm</td>
</tr>
<tr>
<td>Crack opening</td>
<td>Mm</td>
<td>Mm</td>
</tr>
<tr>
<td>Crack opening</td>
<td>Mm</td>
<td>Mm</td>
</tr>
</tbody>
</table>
3.2 Test result and minimum performance requirement

The investigation on the flexural test of notched panels with Dramix® steel fiber and PP fiber has been carried out for a specific project Hydro project in China Jinping II.

The results of this test program conducted at Dalian University of Technology (Ding, 2009) are mentioned below (see Figure 9):

The grade of the plain concrete was designed to be C30/37 on cast concrete

The dosages of steel fibres type Dramix 3D 65/35BG were 20kg/m³ (SF20) to 30 kg/m³ (SF30) 40kg/m³ (SF40), and the macro-synthetic fibre content was 6 kg/m³ (PP6)

![Figure 9](Image)  

**Comparison of flexural strength in the FRC compared to CMOD**

The flexural strength was improved with the addition of fibres. Compared with the SF 20 mix, the flexural strengths of the SF 30 and SF40 mixes increased by 18.1% and 28.2%, respectively. A SFRC panel with greater fibre content indicates a higher load carrying capacity after the incidence of first cracking. The addition of fibres also helps the panels to maintain a better residual load carrying ability.

The flexural strength of the Macro synthetic fibre PP6 mix was similar to that of steel fibre SF20, but after first cracking the load bearing capacity of the PP6 mix dropped by about 60%. This means that the PP fibres have a lower influence on the residual strength than steel fibres. The addition of fibres can increase the energy-absorption capability of concrete panels and this benefit increases with an increase in the fibre content. The improvement in energy-absorption provided by the steel fibres is stronger than that of the macro-synthetic fibres in this trial.
The first draft of the New Model code, 2010, criterion is defined by $f_{R1k}/f_{lk}$ where:

- $f_{R1k}$ = characteristic residual strength at CMOD = 1.0 mm and
- $f_{lk}$ is characteristic flexural strength at first crack.

The draft of the code states that fibre reinforcement can substitute (also partially) conventional reinforcement at ultimate limit state if $f_{R1k}/f_{lk} > 0.4$.

### 3.3 Test result example from job site

For the Violay Tunnel the requirement was 700 joules mini at 25 mm considering a concrete class C30/37. The samples have been spray on the job site and test in Sigma Béton Laboratory.

![Graph showing load vs. deformation](image_url)

**Figure 10**: EN EN14488-5 plates of Dramix® 3D65/35BG 25kg/m³ Curve load –deflection –Sigma report

This curve also show that the $F_{\text{max}}$ (Load max) is always higher than $F_1$ (first crack) which is a key parameters to follow. The load-displacement curve indicates that during the test several cracks are developed. The steel fibres bridging the cracks are generating a perfect load distribution. Once the peak has been reached and the maximum load redistribution effect has been realized, the fibres are being pulled out. Fibre shape and steel strength determine whether the fibres will break or preferably will be pulled out.

Mean value three curve at 25mm => 875 Joules > 700 joules

A specific additional test program was done during the prequalification procedure on job site in order to evaluate the residual strength.

For the residual strength the test method proposed is three point bending test on square panel with notch (EFNARC test procedure).

This test provides additional information about the material properties and allow to check if we meet minimum toughness required by main international standard at different deformation.

The purpose of the different deformation levels is to give flexibility to the designers in the choice of the required deformation of the sprayed concrete under service conditions.

This test was also realized to check the performance criteria of polymer fibre versus steel fibre and the conformity with some minimum requirement.
The Model Code, edited by FIB in 2010, states that fibre reinforcement can substitute (also partially) conventional reinforcement at ultimate limit state if \( f_{R1k}/f_{lk} > 0.4 \).

Dramix steel fibre (anchorage with hook end, E module > 200 00 Mpa, tensile strength > 1300 MPa) play a positive role from early age to hardening concrete.

### 4. Product specification

A new ISO 13270 standard has been published in 2013.

The new ISO standard is important for FRC for following reasons:

1. Reference document for steel fibres for concrete
2. In countries where EN 14889-1 is not applicable, or in countries where no other national standard on fibres is issued, this standard can be taken by the specifier
3. National standard commissions can use the ISO 13270 standard as a blue print for the next national standard.

Some important point mentioned by this standard:

Tolerances on diameter and length: 2 classes are prescribed: more relaxed class B, the same as EN 14889-1 standard, and more stringent class A

Example of a fibre in class B Nominal length of 60 mm can be between 54 and 66 mm Nominal diameter of 0.75mm can be between 0.666 and 0.825 mm.

- An L/D 80 fibre can in reality be a 65. (54/0.825) but according the ISO class B they can still call it a 80 class.
- Therefor it is essential to stress the importance in specifying that the tolerances of the fibres must be according class A
This ISO standard confirm also the following: Steel fibres are suitable reinforcement material for concrete because they possess a thermal expansion coefficient equal to that of concrete, their Young’s Modulus is at least 5 times higher than that of concrete and the creep of regular carbon steel fibres can only occur above 370 °C.

5. Conclusion

FRS is an important high performance method of ground support. Quality and safety can be achieved using the relevant product for the right use. The use of the finished material should be considered along with the test and performance criteria (i.e. match crack widths in the test to expectations in the project and durability requirement).

The fibre type choice and their dosing are determined by the project’s performance requirements during the prequalification test and the proper understanding of the material properties.

Different standards and testing methods of characterisation are available to specify and check the minimum performance of the sprayed concrete reinforced with FRC taking in account Energy absorption, residual strength and creep requirement for each project.

Energy Absorption based on EN14488-5+ Residual strength based on the EFNARC three point bending test on square panels with notch and obtained from a spray panel (600 mm, 600 mm, 100 mm) will be an easy, faster and cheaper way to provide relevant material properties knowledge and quality control.

These two testing methods that use the same moulds can easily be implemented on the job site.

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Ground support and reinforcement system remediation at the Cethana power station, Tasmania, Australia

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D. S. Pennington, Hydro Tasmania, Australia
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Abstract
One of four underground hydro-electric power stations in Tasmania, Australia, the Cethana Power Station within the Mersey-Forth Scheme was constructed in the latter half of the 1960's. Excavation was substantially complete by 1969, with the cavern roof having been established and supported by December 1967. The asset was commissioned in 1971.

As part of its ongoing program of asset management the owner and operator, Hydro Tasmania, had scheduled a major outage to upgrade the power generation equipment in 2015. Prior to that upgrade an extensive geotechnical investigation of the underground excavations was undertaken in 2013. That investigation highlighted that much of the ground support and reinforcement system, which had been installed at the time of initial excavation, had degraded significantly over time and was rapidly approaching the end of its serviceable life. It was assessed that a substantial remediation program should be undertaken to upgrade and/or replace the ground support and reinforcement system throughout the underground excavations prior to the power generation upgrade.

The Cethana Power Station was excavated before any of the rock mass classification schemes and empirical ground support and reinforcement design tools commonly used in the 21st Century had been proposed. None-the-less, the contemporary geotechnical investigation found that the extensive ground support and reinforcement system that had originally been installed closely matched that which would have met current standards.

The remediation program included the replacement of rock bolt reinforcement and shotcrete in the station access tunnel, replacement of rock bolt reinforcement and installation of corrosion protected mesh in the penstock access tunnel, replacement of the rock bolts securing the gantry crane beams to the walls of the power station chamber and replacing the rock bolt reinforcement in the roof of the power station chamber. Additional minor works were undertaken elsewhere.

It was a requirement of the remediation program that the upgraded ground support and reinforcement system have a design life exceeding 70 years. That requirement called for the installation of double-corrosion-protected reinforcement elements. Installation of the system was accompanied by an extensive QA/QC program including shotcrete testing, borehole gauging and photography, grout testing, rock bolt installation tests and pull-testing.

The remediation program was scheduled for implementation over a 6 month period between May and October, 2014. Despite the extent and intrusive nature of the work undertaken, the remediation program was completed successfully, on time, and incident free. It was necessary that this was undertaken without material interruption to power generation throughout the course of the work. It was also necessary for the program to be undertaken in parallel with work necessary to prepare for the power generation outage in 2015. The authors are unaware of any similar ground support and reinforcement remediation program having been implemented in such circumstances.
1 Introduction

The underground Cethana Power Station is located in the steep sided gorge of the Forth River which transects the Fossey Mountains 185 km northwest of Hobart in Northern Tasmania, Australia. The power station complex was excavated in conglomeratic quartzite in the latter part of the 1960’s, with the majority of the excavation occurring in the period from 1965-1968. Construction of the adjacent Concrete Faced Rockfill dam and the installation of power generation equipment was completed and commissioned in 1971.

The Cethana Power Station (Figure 1) was excavated in a sequence of generally northeast dipping quartz pebble conglomerates and sandstones of the Roland Formation which originally formed as an extensive series of fluvial fan deposits across what is now northern and western Tasmania. All facies within the geological sequence at Cethana have undergone regional metamorphism to quartzite. However, the nature of the strata varies, with those beds derived from conglomerate being significantly more massive than the sandstone beds. The general nature of the stratigraphy at Cethana is a sequence of massively bedded conglomerates which grade up to more thinly bedded sandstone facies. The sandstone beds are typically sharply truncated by an overlying but conformable conglomerate which in turn grades to sandstone. The stratigraphy appears to repeat over a thickness of 20-40 m with conglomerate facies comprising up to 80 percent of the sequence.

![Figure 1](image)

Figure 1 Location of (left) and access to (right) the Cethana Power Station in Northern Tasmania

Excavation of the Cethana Power Station occurred during the first decade of rock bolting for permanent support of underground excavations. As described by Mills (2009) research and development to achieve permanent rock bolting was carried out by the Snowy Mountains Hydro-Electric Authority in New South Wales and led to the completion of the Tumut 2 Underground Power Station as the first excavation of its type in 1961 (Pender, Hosking and Mattner, 1963). The key to permanent rock bolting was encapsulation of the rock bolt in Portland cement grout for corrosion protection. A development in the 1960’s was modification of the bearing plate to include grout delivery and air breather tubes. The rock bolts themselves continued to be a mechanical anchor and the grout was not considered to be a bolt retention or load transfer element. The Hydro Electric Commission in Tasmania embraced this slot and wedge grouted rock bolt technology which were used extensively at underground facilities prior to their use at Cethana. The specific methodology employed in Tasmania was described in some detail by Colebatch, Endersbee and Paxton (1963) in their discussion of the Pender, Hosking and Mattner (1963) paper and the modified bolt is illustrated in Figure 2.
In addition to the rock bolts, support of the Cethana Power Station cavern included the use of standard shotcrete and/or mesh. The use of reinforced concrete for ground support was confined to portals.

In 2013, after 44 years of operation, the Cethana Power Station was being prepared for a major refurbishment of the turbine generator to be undertaken in a power generation outage in 2015. At that time a full geotechnical assessment of the dry underground excavations was undertaken with the aim of determining the condition of installed ground support and reinforcement elements, characterising the rock mass using currently accepted empirical techniques and identifying the requirements of a remediation program sufficient to ensure the integrity of the asset for a further 70 years or more.

2 Geotechnical investigations and assessments

Geotechnical assessments commenced with overall inspections of the dry underground excavations. These included the access tunnel, the station cavern (with emphasis on the anchorage of beams supporting a gantry crane and reinforcing the roof of the cavern), the penstock access tunnel, the ventilation tunnel and the fan room. The inspections identified a number of defects attributed to the age of the excavation. This included degraded shotcrete (not fibre reinforced), particularly in the access tunnel and corrosion of rock bolt plates where they were exposed. The primary agent of degradation and corrosion was acidic groundwater due to oxidation of detrital pyrite within the rock mass.

2.1 Rock mass characterisation and kinematic wedge analysis

Investigations were undertaken to determine if it would be necessary to replace the ground support and reinforcement system.

Rock mass characterisation using the Q system of Barton, Lien and Lunde (1974) was cross-checked with the rock mass rating (RMR) system of Bieniawski (1973). Excavation of the Cethana Power Station had preceded the development of these now commonly used empirical tools. Data from the original geological investigations by Hale (1964) and Rawlings (1965) was available. This was truthed with scanline mapping of exposed rock in unshotcreted tunnels and from outside the station portal. Drill core from historical geotechnical investigations was not available for logging. Q parameters were determined using the method of Palmström (1982) for rock quality designation (RQD), reference to NGI (2013) for the joint
parameters and Kirsten (1983) for the stress reduction factor (SRF). The conclusion reached was that in terms of Q parameters, excavation support ratio and excavation span or height, the roof of the access tunnel and station cave plotted as “Fair” ground and the walls plotted as “Good”. The rock mass rating (RMR) system was consistent with the Q system assessment. The span ratio for the cavern and access tunnel roof and walls are plotted against Q and overly the Q Chart in Figure 3.

Figure 3  Q Chart (NGI, 2013) for the Cethana Power Station

A further check on the adequacy of the installed support was a kinematic wedge analysis of mapped joint sets. The Rocscience program Dips was used to analyse the data and wedge analysis was undertaken using the program Unwedge to determine the factor of safety of the historically installed rock bolt pattern. The largest feasible wedge had a mass of 120 tonnes, a supported height of 3.4 m and a factor of safety of 2. These checks confirmed that the ground support and reinforcement system installed at the time of excavation would be appropriate even if the complex was being excavated today.

2.2 Rock bolt integrity testing

A program of non-destructive testing (NDT) using vibrational responses to an impact on the head of the bolt was carried out on exposed rock bolts throughout the complex to ascertain whether the corrosion evident at excavation surface had also occurred within the rock mass. This testing was used to predict that in excess of 80 percent of the rock bolts had a long free (or unbounded) length indicating that most of the rock bolts were either not fully grouted or were excessively corroded.

To quantify these indications from the NDT, a sample of suspect rock bolts was over-cored. The overcoring showed that the historical grouting of the rock bolts had only partially encapsulated most of the bolts, and any grout present had typically only grouted the first 150 mm of the annulus around the rock bolt, creating a localised plug. Where the rock bolts were not encapsulated, isolated pitting corrosion was evident, and was attributed to the acidic groundwater conditions.
A sample from the rock bolts shown to be apparently fully grouted by the NDT was pull-tested to 10 t. The majority held this load, however, due to the evidence of significant corrosion it was decided that all the rock bolts would have to be replaced.

2.4 Shotcrete condition assessment

Inspections showed that the shotcrete installed in the station cavern was generally in good condition. A few cracks were evident and one of those in the roof had given some cause for concern in the past, albeit that those investigations had not recommended any remedial work. A horizontal crack in the shotcrete on the north wall of the station cavern provided evidence that the rock bolts supporting the crane beam on that wall were distressed. Yet while remedial work would be required on the rock bolts, the shotcrete was adequate.

The shotcrete in the access tunnel however, was extensively degraded. It was found to be less than 10 mm in thickness in a number of locations and had also delaminated from the rock in some areas. General replacement was therefore required.

3 Remediation ground support and reinforcement design

3.1 Overview

Cethana Power Station was not a new project. As discussed above, it was known that:

- The original rock bolts had been installed between 45 and 47 years prior.
- The rock bolts over-cored during investigations were ungrouted over most of their length.
- The majority of rock bolts exhumed exhibited isolated deep pitting corrosion yet 60 percent of those load tested exceeded a load of 100 kN.
- Where over-cored rock bolts were fully encased in grout (typically for 150 mm up from the collar) there was negligible evidence of corrosion.
- The majority of rock bolts were found to be holding on their slot and wedge mechanical anchor.

Various authors including Baxter (1996), Pells and Bertuzzi (1999), Adams, Lechner and Lamb (2001), Bertuzzi, (2004) and Shen and Chan (2008) have discussed the longevity of reinforcement elements and suggested that up to 100 years could be achieved. All those authors were working in the Hawkesbury Sandstone in New South Wales, which is a significantly more benign environment than the acid generating hard rocks at Cethana. Given this, and the performance of the effectively ungrouted rock bolts at Cethana, it was judged that a design life of 70 years is reasonable.

3.2 Double corrosion protected rock bolts and cable bolts

Two variations of double corrosion protected (DCP) reinforcement elements were used during the remediation works: DCP rock bolts™ and flexible DCP-Ten cable bolts™. Both were sourced from Dwyidag Systems International (DSI).

The reinforcement elements were to be fully grouted into the hole with no free length. While the toe and collar fittings were expected to provide an element of load bearing capacity, the reinforcement elements were designed to act as a dowel with load being transferred to the ground along the length of the bolt.

The DCP rock bolt™ (hereafter rock bolt) was a 20 mm galvanised rock bolt with a design yield strength of 240 kN. It was covered by a loose fitting polyethylene sleeve which protected it from contact with the rock mass and importantly, provided a second level of corrosion protection. Initially the rock bolt was secured by an expansion shell anchor at the toe. The polyethylene sleeve was also designed to act as the grout delivery tube, with grout entering through a specialised grouting bell and returning down the annulus outside the sleeve.
The 23.5 mm DCP-Ten cable bolt™ (hereafter cable bolt) was designed as a flexible version of the solid DCP rock bolt comprising woven strands giving a design yield strength of 500 kN. The cable bolt comprised an unsheathed section at the toe which incorporated three Garford (birds nest) bulbs. During installation, this section of the bolt was fully encapsulated in resin allowing the cable bolt to support load a relatively short time after installation, prior to the remainder of the bolt being post grouted through a polyethylene tube in similar fashion to the rock bolts. Cable bolts were chosen for reinforcement of the cavern roof over rock bolts for two compelling reasons:

- Their flexibility enabled the required 4.2 m long elements to be installed in the confined 1.7 m headroom of the ceiling cavity without the necessity to remove the ceiling cladding; and
- The superior yield strength allowed the requirement to be reduced from over 200 elements to 90.

3.3 Locations identified for remediation at Cethana

The ground support and reinforcement of the rock mass identified for remediation is outlined below.

1. The access tunnel: 2.4 m long rock bolts at an angle of +75° and vertical on a 1.2 m x 2.5 m pattern in the roof. High strength fibre reinforced shotcrete applied above the invert throughout to a minimum thickness of 30 mm.

2. The penstock access drive: a similar pattern of rock bolts to the access tunnel in the roof, with epoxy-coated (for corrosion resistance) heavy gauge mesh as surface support in the roof.

3. Crane rail beams: replacement of the 6.0 m long rock bolts securing the 25 m long crane rail beams at 1 m centres.

4. Cavern Roof: A vertical pattern of rock reinforcement commensurate with the capacity of the original reinforcement pattern in the roof of the station cavern. The extant mesh reinforced shotcrete was found to be adequate to compliment the rock bolting.

Other associated work included re-anchoring the suspended false ceiling of the station cavern. The existing shotcrete support of the cavern walls was adequate.

4 Challenges associated with the remediation programme

4.1 Operational challenge

Rock falls could cause injury and damage and were key safety risks to personnel and production. It was thus necessary to restore the structural integrity of the cavern complex prior to the outage, and provide a safe environment for the numerous personnel refurbishing the machine. Furthermore, completing the cavern rock bolting works over the existing machine avoided exposing the refurbished machine to any risk of damage.

Hydro Tasmania has a demanding $70M-$80M per annum capital refurbishment programme embodied in a rolling ‘10-Year Plan’. Any delay would compromise the ability to reduce and manage risk consistent with the Plan. Consequently, the unplanned cavern works became an additional site establishment activity for the machine refurbishment project to be undertaken concurrently with the others. These were mainly accommodation and facility building works and not insignificant. This presented a significant logistical challenge.

Cethana power station is integral to the Mersey Forth Power Development, which is a key run-of-river scheme in Hydro Tasmania’s portfolio. Consequently, the machine had to remain in service as far as possible throughout the works. Since the consequences of a rock or drilling slurry falling onto an operating machine would be significant, effective protection of the machine and sensitive equipment became paramount.
Essentially the project involved undertaking contracted rock bolting and shotcreting activities — i.e. major civil underground works - in an operating power station.

A further complexity was that this was the first time since hydropower construction ceased in Tasmania that a project of this nature was undertaken. A local mining contractor experienced with development-type projects was deemed the most suitable for the work.

The undertaking became a complex and challenging project, requiring rigorous planning, management, coordination and collaboration.

4.2 Construction challenge

The programme of work was scheduled into the following four stages:

- The access tunnel
- The penstock access tunnel
- The crane rail beams
- The station cavern roof

4.2.1 Access tunnel

The methodology to refurbish in the access tunnel was relatively straightforward:

- The tunnel was pressure washed to remove significant build up of ferric hydroxide and sulphur salts and areas of thin detached shotcrete;
- Degraded and debonded shotcrete was scabbled from the walls and roof using a standard development jumbo that worked back from the cavern towards the portal;
- The rock bolts were installed in the roof commencing from the portal; and
- The shotcrete was applied on retreat to the portal.

4.2.2 Penstock access tunnel

Remediation of the penstock access tunnel was similar to the access tunnel, despite it having been previously unlined and only spot bolted. The rock bolts and mesh were installed along the length of the tunnel. Access restrictions required the work be carried out using hand held equipment working from a scissor-lift platform.

4.2.3 Cavern crane beams

A fifty tonne gantry crane is supported on two reinforced concrete beams anchored to the long walls of the cavern. While the method of anchorage is not standard practice in similar installations today, the decision was taken to replace the existing anchors using a similar approach to the original installation. Geotechnical aspects of the work entailed:

- Accurate drilling of bolt holes at an inclination of 30° above horizontal to permit the replacement of existing rock bolts in the angled face of the crane rail beams;
- Installation of 6 m rock bolts between every second pair of existing degraded 20 ft slot and wedge rock bolts, and;
- Installation of extensometers in both crane beams on the centre-line of the turbine and adjacent to the access tunnel to monitor behaviour during crane operation into the future.

Bolt hole drilling was undertaken as a combination of diamond drilling through the concrete mass of the beam, followed by percussion drilling to a total final depth of 6.1 m to facilitate installation of the bolts. The
aim of the two pass drilling was to minimise damage to the structural beam by minimising the risk of micro-shattering the concrete. The diamond drilling also had the advantage of greater position and alignment accuracy, which when combined with careful cover meter location of the essential front reinforcement bar, reduced the risk of compromising the structural capacity of the beam.

An additional challenge was the requirement to reliably capture and contain all drilling water, which was much simpler with diamond drilling at the collar of the hole. The methods used to capture the water for both the coring and percussion drilling operations are illustrated in Figure 4. A total of 28 bolts were installed for each crane beam.

![Image](image.png)

**Figure 4** The diamond drill set-up for drilling the crane beams (left) and the percussion drill mounted in a work basket slung from the crane (right)

Working around the high-voltage station bus bars presented a significant safety hazard. Exposure was avoided by working in this area only when the station was shut down, which occurred occasionally for short periods. This required close coordination with the generating-system controllers.

### 4.3.4 Cavern roof

As previously mentioned, the ceiling space headroom for installing the cavern roof reinforcement was typically less than 1.7 m - a challenging operating space. In such a constrained space, flexible cable bolts allowed installation of much longer bolts than would have been possible using conventional rigid rock bolts without removing the ceiling cladding. This was a crucial advantage, since removal of the ceiling cladding would have required shutting down the turbine due to the high risk that ground and drilling water normally diverted by the ceiling would threaten the alternator. While the required cable bolt length based on kinematic analysis matched the original requirement, the capacity of the 4.2 m long cable bolts enabled a more open bolting pattern, reducing the number of bolts that would otherwise have been required to provide the same level of support.

There were also challenges to providing safe access to the ceiling cavity in order to rehabilitate the cavern roof. These included:

- The existing access walkway to the ceiling space had been downgraded to a safe working restriction of three people with hand tools because it was suspended from the roof by the same type of rock bolts that had been found to be compromised by corrosion at a number of locations.
- The load on the ceiling framework to carry out the drilling exceeded its capacity.
- The ceiling itself comprised sheets of colour-bond steel screwed from underneath and was not rated to carry additional loads.
- Access to the ceiling cavity, 16m above the assembly bay floor, was limited.
The special measures developed to overcome these challenges are partly illustrated in Figure 5. A moveable working and rock fall deck was installed on the suspended ceiling structure in the roof cavity. This deck was supported by a scaffold frame erected on the gantry crane. This combined arrangement was moved forward as the work progressed. In addition, a scissor-lift platform was used to raise tools and equipment to the ceiling space.

Load and access restrictions remained in force for all areas not rehabilitated or directly supported by the scaffold. Pedestrian access was via a scaffold staircase at one end of the ceiling space and a ladder for emergency exit at the other.

Working in the limited headroom space also posed significant ergonomic challenges. Solutions included frames and offset controls for the drill rigs along with seats for drill operators.

![Figure 5](image)

**Figure 5** Scaffold on the crane gantry supporting the ceiling structure (left), the working deck laid on the purlins of the ceiling structure (centre), and a drill operator sitting on a stool in the low headroom (right)

Figure 6 illustrates installation of the cable bolts within the confined space.

![Figure 6](image)

**Figure 6** Cable bolt installation (left), and grouting (right)
5 Development of QA/QC program to achieve grout integrity assurance

5.1 Installation trials

The rock bolt and cable bolt installations undertaken in remediation of the Cethana Power Station were critical to the success of the program. It was essential that each installation was undertaken with a high level of scrutiny and that everyone involved was confident that high quality installations had been achieved.

The installation of the cable bolts with double corrosion protection to provide the design life required was unproven. Given that the elements chosen were a relatively new variation to established cable bolting practice and experience in their use was limited, grouting trials were carried out to prove their suitability.

Initially, trials were conducted to ensure that full encapsulation was achievable for each length of each type of reinforcement element at the orientation at which it would be installed in the ground. Consequently several samples of each were grouted into a PVC conduit or steel pipe of similar diameter to the installation holes mounted in the yard as they would be underground. Rock bolts were held by the expansion shell anchor and cable bolts by resin. Grout was mixed to pre-determined water:cement ratio of 0.33-0.35 using a paddle mixer and delivered to the test elements using a piston pump. The number of piston strokes required to achieve a grout return was recorded, and several additional piston strokes occurred before the grout delivery was shut off. The average number of pumps formed a benchmark for the respective elements to ensure full encapsulation when they were installed underground.

5.2 Trial results

5.2.1 Rock bolts

The grouting trials to simulate the installation environment for 2.4 m and 6.0 m rock bolts were successful. After the grout had set for 24 hours, the test samples were cut a number of times and demonstrated full encapsulation in all cases.

5.2.2 Cable bolts

Despite the difficulties of mixing the resin in the test tubes, the results were quite encouraging with good encapsulation and minimal gloving.

Initially, cement grouting was a failure. While the inner annulus and the collar end of the outer annulus of the polyethylene sleeve could be fully grouted, inspection showed the toe end of the outer annulus of the polyethylene sleeve was not grouted. It was concluded that as the cross sectional area of the outer annulus relative to the inner annulus through which the grout was being delivered was too great. As a consequence, the grout lost pressure when it exited the inner annulus and fell back to the collar under gravity. Ultimately this prevented air bleed and left the toe section of outer annulus of the cable bolt ungrouted.

The Contractor suggested that a way to overcome the problem would be to grout the cable in reverse by delivering the grout via the outer annulus and allowing the grout to return down the inner annulus of the polyethylene sleeve and out through the grouting bell. By this method it was surmised that the grout would remain under pressure while flooding to the toe of the hole and returning to the collar through the grouting bell. The method required a 22 mm hole to be drilled through the dome section of the bearing plate for the insertion of the grout tube. For the purpose of the trial, a 250 mm section of 68 mm pipe was welded to the 53 mm test pipe to accommodate the grout tube. During installation underground, this requirement would be accommodated by reaming the first 200 mm at the collar of the installation hole.
The modified trial was satisfactory and the Supplier concurred with the result stating that the intent of the installation was not compromised. Figure 7 illustrates the results of the two trials.

Figure 7  Initial trials for grouting cable bolts were unsuccessful (left), however reversing the grout flow provided a satisfactory result (right)

It was concluded that the modified method of grouting the cable bolts would result in a satisfactory installation. Moreover, the modification demonstrated a willingness by all parties to develop a practical solution without compromising quality and performance.

5.3 Installation QA/QC

In addition to grouting trials, a number of protocols were developed to ensure that quality installation was achieved. These included:

- Trial drilling and borehole gauging to determine bit life and ensure that consistent hole clearances were maintained (Figure 8),
- Borehole flushing at the completion of drilling to ensure that all cuttings were cleared.
- Borehole camera survey to confirm that the holes were clean and that no rocks and rubble would impede the installation and also to record the incidence of structures and cavities that might consume more grout,
- Careful measurement of borehole length so that the void to be grouted was known, and particularly in the case of the cable bolts to ensure that the resin cartridges would not be lost in the toe of the hole, and
- Logging the number of pump strokes taken to fully grout the cable bolts with the understanding that if insufficient strokes were required, then the sheath may be compromised and that if excessive strokes were required that voids potentially remained around the cable bolt. Where this occurred, the bolts were replaced.
The rock bolt specifications provided by the supplier recommended a 43-45 mm hole. In the example illustrated, that range was slightly exceeded but not materially so. Critically, holes drilled with new bits and old bits up to six holes per bit, exceeded the minimum diameter ensuring adequate encapsulation of the bolts when grouted. Consequently, it was considered that drilling up to six 2.4 m rock bolt holes per bit would provide an adequate hole for the correct installation. Similar trials were completed for all installation combinations.

### 5.3.1 Borehole camera surveys

Boreholes for the installation or rock bolts, cable bolts or extensometers (discussed below), and any which were ultimately abandoned for any reason, were surveyed using a borehole camera prior. The camera was capable of taking a video of the hole with voice overlay as the camera was inserted. The camera also had the ability to take still photographs as required. The purpose of the surveys was to inform the entire installation process including:

- Pre-warning of hazards which might preclude the satisfactory installation of a given rock bolt or cable bolt including the presence of rubble (Figure 9), voids, old steel etc,
- An indication of features such as drilling voids or natural cavities (Figure 9), which might impact the volume of grout required in a particular case,
- Determining the optimum position for the location of instrumentation with respect to pre-existing geological structure, and
- To provide a database from which a structural model of the rock mass could be constructed at completion.

![Figure 8 Bolt hole gauging of holes for 2.4 m rock bolts in the access tunnel](image-url)
5.4 Rock bolts testing regime

5.4.1 Non-destructive (NDT) testing

A program of non-destructive testing was conducted on a randomly selected sample of rock bolts, except for the Crane Beam bolts where all were tested. The analyses of the responses enabled a rating to be assigned to each bolt as follows:

- Rating 1 - good installation with minimal free length
- Rating 2 - potential gap grouting, and
- Rating 3 - long free length, poor grouting

The testing provided a qualitative measure of the integrity of the installation, and consequently is not definitive.

The majority of the tested samples returned Rating 1, with some Rating 2 results on the crane rail beam bolts. A likely explanation for the latter is that grout escaped at the crane rail beam / rock interface leading to differential encapsulation tightness. It was reasoned that in this case the bolt itself would still be fully encased in grout, and corrosion protection would not have been compromised.

5.4.2 Grout testing

The QA/QC program for rock bolt grouting required the determination of Unconfined Compressive Strength (UCS). This was done indirectly by carrying out point load testing at 7 and 28 days, with required strengths of 32 MPa ± 10 percent and 45 MPa ±10 percent at 7 and 28 days respectively. The former was achieved for all batches and the latter was achieved for all but one batch which returned a UCS of 44 MPa. This was considered acceptable.

5.4.3 Pull testing and proof loading

5.4.3.1 Pull testing

The Technical Specification provided for pull-testing of 15 percent of installed rock bolts in the Access Tunnel and Penstock Tunnel, to a load of 10 tonnes after a period of 7 days. A pull-testing pattern was agreed prior to the commencement of testing to prevent an observational bias in the selection process, and bolts were tested by the Contractor and the Principal Representative working together.
5.4.3.2 Proof load testing

It was a requirement of the Technical Specification that a total of 10 Crane Beam rock bolts be proof loaded to 20 tonnes for a period of one hour. Although it was originally expected that proof loading would take place after 28 days, this work was undertaken after 7 days when the average 7 day strength was demonstrated to be well in excess of the anticipated 32 MPa. This change provided significant benefits to the scheduling of the works. All bolts tested satisfactorily with no slip being recorded.

5.5 Shotcrete

5.5.1 Access tunnel

The shotcrete mix design was tailored to meet the high strength requirements. Quality tests were undertaken to ensure that the shotcrete achieved the design strength. With the Cethana Power Station continuing to generate power during the works, there was always the possibility, albeit unlikely, that access to the station by maintenance personnel with equipment could be required at short notice. Consequently, there was also a requirement to monitor the gain in strength of the shotcrete as it was applied so that safe access for maintenance personnel could be granted as soon as possible. The extensive monitoring program included:

- Slump tests at the batch plant prior to the 50 km one hour journey to the site and underground prior to application with a target of 120 mm;
- Combined needle penetrometer and mini beam tests using the methodology described by Clements (2004) to monitor early strength development of the shotcrete with a quasi-uniaxial compressive strength (UCS) target threshold of 1 MPa set for re-entry by personnel should it be required (fibre reinforced shotcrete was applied on retreat and therefore early re-entry by those undertaking the work was not a requirement). In general safe re-entry could have taken place at 8 hours;
- UCS determined for the fibre reinforced shotcrete using a range of increasing simpler techniques for wider coverage. These included:
  - Two batches from which cast cylinders were collected, compacted, stored and prepared for laboratory testing in accordance with AS1012. Average strengths \( f'_c \) of 37.5 MPa at 7 days and 53.0 MPa at 28 days were measured;
  - Eight batches from which samples were cored from sprayed test panels and tested in the laboratory; a probabilistic determination of shotcrete strength using the methodology of the Concrete Institute of Australia (2002) anticipated the UCS for cored samples \( f'_cc \) loaded axially, would be of the order of 15% less than the target UCS of cast and compacted samples. A 95 percent probability of \( f'_cc > 34 \) MPa was satisfactorily achieved;
  - A number of small diameter cored samples from the same eight panels tested by point load test to AS4166.4.1.
- Shotcrete thickness testing by random drill testing in the roof and each wall at 10 m intervals along the length of the access tunnel. Shotcrete bond tests were not undertaken.

The refurbished access tunnel is illustrated in Figure 10.
5.5.2 Cavern roof

During original construction, a nominal 75 mm thick shotcrete layer was sprayed over weld-mesh secured to the roof by the original rock bolts. While it was not the intention to replace this shotcrete, there was some concern over its integrity due to observed cracking and the action of acidic ground water which had compromised the original rock bolts and led to a build-up of ferric hydroxide and sulphur salts in some patches. The integrity of the shotcrete, its adherence to the rock mass and thickness was confirmed by sounding and drilling of the rock bolt holes. Prior to any drilling, a cementitious polymer of high tensile strength and very good adhesion over a typical sprayed thickness of up to 4 mm, was sprayed over the shotcrete to provide a long term seal against degradation due to atmospheric conditions and protect exposed steel from corrosion. Surface preparation was limited to the requirement to hose off dust and dirt.

5.6 Instrumentation and monitoring

5.6.1 Crane rail beams

Paired resistance wire extensometers (RWE’s) were installed in both Crane Beams on the centre line of the Access Tunnel rail tracks and the centre line of the Machine. At each site a 1 m RWE was installed across the Crane Beam / rock interface and a 3m RWE was installed between the Crane Beam and the deeper seated rock mass. The general arrangement is illustrated in Figure 11.
The instruments were temporarily commissioned with a stand-alone data logger prior to proof loading of the Crane Beam rock bolts. While no perceptible response occurred in most cases, a very slight response was observed on the northeast instruments coinciding with proof loading of some bolts. Put in context, the displacement on the 3 m instrument was of the order of 0.06 mm and that on the 1 m instrument approximately 0.4 mm. Ongoing long period fluctuation of the 3 m instruments occurred as a result of temperature fluctuations in the Cavern. The instruments themselves do not incorporate a temperature sensor or temperature compensation system as the impact of temperature change on the inhomogeneous rock mass is indeterminable.

5.6.2 Cavern roof

Similarly, a total of eight 3 m RWE’s were installed in the Cavern Roof to monitor the stability of the rock mass. The location of the instruments was determined from detailed scrutiny of video footage from cable bolt and rock bolt boreholes. In addition to the extensometers, an array of survey prisms was installed for long term monitoring of existing cracks in the shotcrete and the excavation generally.

5.6.3 Structural modelling

Utilising the borehole videos and photographs, significant structures were delineated to create a 3D model of the rock mass. Despite some logistical complication arising from the empirical nature of the data, a log of structures was prepared and major discontinuities and structures transferred to a 3D model of the completed drill holes. Hole-to-hole interpretation of planes was then undertaken with reference to the structural geology of the cavern roof as mapped by Godfrey (1970) in an endeavour to "ground truth" the outcome. Figure 12 shows a view of the Cavern Roof illustrating the location of all boreholes, identified structural planes and the location of the extensometers. The gross structural geology of Godfrey (1970) is traced over the plan. It is not possible to state that the model is definitive. However, it is a reasonable representation that honours the original mapping in a general sense.
6 Conclusion

The remediation of the Cethana underground cavern complex was completed successfully, safely, on time and within budget. It prepared the station for the planned machine refurbishment in 2015. It is particularly significant that the entire remediation program was undertaken with negligible interruption to power generation. Only one shutdown of 90 minutes was required to facilitate the work. We are not aware of any similar remediation program undertaken worldwide in an underground power generation facility while the power generation equipment remained fully available.

The positive outcome was the result of fit-for-purpose design, thorough planning, rigorous QA/QC and close collaboration of the project team. The project restored the integrity of the underground cavern complex as a safe workplace and will enable renewable energy production at the site for another 70 years or more.

Acknowledgement

Work on the remediation of the Cethana Power Station was undertaken under the management of Hydro Tasmania. Geotechnical investigations, ground support and reinforcement system design, specification and quality assurance and control were undertaken by consultant engineers Pitt & Sherry. Mancala was engaged as the works contractor and provided active input to the design in practice. All parties worked together throughout the course of the program to deliver the final objective within specification, on time, on budget without incident and without interruption to the normal operation of the power generation asset.

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Ground support and reinforcement system remediation at the Cethana power station, Tasmania, Australia

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Ground Support for Mining Through Weak Graphitic Faults at the Casa Berardi Mine, Quebec

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Abstract

Casa Berardi is a gold mine located in the North-Western part of the Quebec province (Canada). The near vertically-dipping, narrow veins are found along graphitic faults. The main being the Casa Berardi Fault which is a several metre wide regional structure with extensive graphite coating on shear surfaces. The host rock mass is primarily sediments, consisting of relatively competent greywackes and weak, heavily-foliated argillites. During early-life production in the 1990’s, the Casa Berardi Mine suffered three major collapses characterized by gravity-induced vertical “running” of the graphitic Casa Berardi fault zone to the ground surface. These collapses resulted in closure of the mine in 1997. The lack of bulking on failure is the inherent characteristic of the fault material, with possible non-bulking at certain locations. Contrary to typical rock material, the volume of broken graphite may stay relatively close to its original volume. This particular characteristic can cause unravelling to significant vertical dimensions if no precautions are used during excavation and mining through the fault. Furthermore, if the generated void is unable to fill due to bulking, the instability can quickly transition into a chimneying disintegration failure mechanism as graphitic material and host rock will cave. The extent of this caving mechanism is highly unpredictable as propagation can be unseen within the rock mass, and can propagate large distances (in excess of 150m) up until it intersects the ground surface, sometimes over a period of years. Since the reopening of the mine, the mining methods include both transverse and longitudinal open stoping, and are very successful despite the challenges in mining both through and along the graphitic fault. This paper focuses on the specific attention to monitoring and support methods for mining within a graphitic fault zone. Firstly, case histories of the fault failures leading to mine closure in early life of production are examined. Secondly, the evolution of ground practice in graphitic fault zone is described. Finally, a general discussion on the operational challenges in respecting the fault extraction specifications is presented.

1 Introduction

The Casa Berardi Mine (CBM) property is located approximately 95km north of the municipality of La Sarre in north-western Québec, Canada (Figure 1). Established on 13 claims that were discovered in 1974, Inco Gold and Golden Knight Resources Inc. formed a joint venture to operate the East Mine and West Mine, with underground operations commencing in 1988 and 1990 respectively (Figure 2). In 1991, TVX Gold acquired Inco’s interest in CBM and operated it until 1997, when the mine was closed as a result of ground control issues related to unravelling of the fault. In total, during the period from 1988 to 1997, 700 000 ounces of gold were produced from the two mines. In 1998, Aurizon Gold acquired the Casa Berardi Mine, but placed only the West Mine back into production. Hecla Limited then acquired Aurizon Gold in June, 2013 and formed the Hecla Quebec division. During this second period of acquisition, the total production from 2006 to 2015 is 2.05M ounces of gold. The mine is currently entering its 10th consecutive production year, the current budgeted production is 131 000 ounces of gold, with an average grade of 5.9 g/tonne and a mill capacity of 2 800 tonnes per day. As of December 31st 2015, the proven and probable reserve are 9.3Mt million tonnes with an average grade of 4.47g/tonne (open pit and underground operations combined), expending the life of mine for an additional 10 years (Hecla Limited, 2016). The current production zones are the 113, 118, 123 and 124, with the East Mine Pit entering production in July 2016.
1.1 Geological setting

The CBM is located in the northern part of the Abitibi subprovince, a subdivision of the Superior Province, the Archean core of the Canadian Shield. The mine area belongs to the Harricana-Turgeon Belt, which is part of the North Volcanic Zone. Structurally, the property is enclosed in the Casa Berardi Tectonic Zone, a 15 km wide corridor that can be traced over 200 km. A network of east-west to east-southeast and west-northwest ductile high strain zones mainly follows the lithological contacts.

The Casa Berardi Fault (CBF), being the main pervasive graphitic fault, is defined by a stratigraphic contact between a graphite-rich sediment sequence at the base of the Taibi domain, a northern continuous intermediary fragmental volcanic unit, and a southern polymictic conglomerate unit composed of weak,
heavily-foliated argillites. On the north side of the fault, a thick sequence of very homogeneous wackes belonging to the Taïbi Group is affected by amphibolites of metamorphic grade. The fault strikes east-west and is subvertical, generally dipping 85° to the south or the north. The main fault is composed of graphitic material which is a several metres wide (0.5m to 5m, occasionally with widths to 10.0m) regional structure with extensive graphite coatings on shear surfaces, constituting auxiliary faults not thicker than 1.0m.

The Casa Berardi gold deposit can be classified as an Archean sedimentary-hosted lode gold deposit. Gold mainly occurs south of the CBF, and sometimes is found on both sides of the fault, in proximity of the auxiliary faults. The mineralization system is composed of large, low-sulphide quartz veins and low-grade stockworks and carbonate-mica replacement zones. Main vein systems are surrounded by quartz veinlet stockworks mostly developed in strongly carbonate and sericite-altered host rocks. In general, gold is associated with arsenopyrite with minor pyrite under the form of a few tens of micrometre free particles, attached grains to sulfides, or locked grains in sulfides in various proportions depending on the mineralized areas.

1.2 Geotechnical characteristics

Several geotechnical campaigns/laboratory testing were performed at the CBM. Below is a summary of the most prominent properties commonly used in geomechanical analysis.

1.2.1 Geomechanical zoning

Due to the variability in the geological lithology, a generalized geomechanical grouping was performed in order to set different design sectors. Figure 3 represents the typical mining plan layout with regards to the affected zoning and the associated average geomechanical classification. As can be noted, four distinct zones are grouped, which are generally separated by a fault contact: (i) Volcanic rock (V); (ii) Wacke (S3); (iii) Graphitic Mudstone (S3S6) with intrusion of Quartz Ore and Pyrite Rocks; and (iii) Siltstone (S4).

![Figure 3 Geomechanical zoning attributed to fault structures (FN1, CBF, FS2), typical plan view for transverse mining.](image-url)
1.2.2 Strength characteristics

The table below lists the typical UCS properties associated to each rock type domain.

Table 1  Uniaxial Compressive Strength laboratory results synthesis

<table>
<thead>
<tr>
<th>Rock Type</th>
<th>Unconfined Compressive Strength</th>
</tr>
</thead>
<tbody>
<tr>
<td>Volcanic rocks</td>
<td>85 MPa</td>
</tr>
<tr>
<td>Wacke</td>
<td>60 MPa</td>
</tr>
<tr>
<td>Graphitic Mudstone</td>
<td>40 MPa</td>
</tr>
<tr>
<td>Pyrititic chert Rocks</td>
<td>105 MPa</td>
</tr>
<tr>
<td>Quartz Ore</td>
<td>100 MPa</td>
</tr>
<tr>
<td>Siltstone</td>
<td>60 Pa</td>
</tr>
</tbody>
</table>

1.2.3 In-situ stress

Using conventional door-stopper technique, stress measurements were performed by Canmet at the West Mine (Arjang & Anderson, 1999). The resultant stress gradient is presented in Table 2.

Table 2  Stress gradient used at the CBM, Canmet (Arjang & Anderson, 1999)

<table>
<thead>
<tr>
<th>Stress, (MPa)</th>
<th>Orientation</th>
<th>Residual</th>
<th>Remark</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\sigma_1$ (E-O) = 1.85$\sigma_3$ (MPa)</td>
<td>Azimuth 75°</td>
<td>Dip -17°</td>
<td>Orebody-E-W</td>
</tr>
<tr>
<td>$\sigma_2$ (N-S) = 1.07$\sigma_3$ (MPa)</td>
<td>Azimuth 171°</td>
<td>Dip -17°</td>
<td>Orebody-perpendicular</td>
</tr>
<tr>
<td>$\sigma_3$ = $\sigma_v$ $\approx$ 0.033d (MPa)</td>
<td>Azimuth 122°</td>
<td>Dip 65°</td>
<td></td>
</tr>
</tbody>
</table>

Where d = depth below ground surface

1.2.4 Graphitic fault characteristics

The graphitic sediments in the fault zone area are a dark grey colour, with the fractures or cleavage being lustrous in appearance - no particular grain orientation can be observed. The actual graphite content in the main fault can reach up to 80%, with auxiliary faults varying between 5 to 50% in graphite content (i.e. zone of altered rock mass, with graphitic material embedded between the foliation planes). The higher the graphite content, the more likely the fault appears to be compacted or consolidated (appears as a lithological intrusion), and tends to be more friable as it contains 10% to 25% swelling clay content. The stand-up time for graphitic material is very low (24hours or less, observed during development), and it appears to have a low bulk factor. Nonetheless, the fault material is relatively impervious and stable in the confined natural state.

Table 3 generalizes the strength parameters for the graphitic fault zone material and for the graphite altered rock (can be associated to the auxiliary fault zones). The lack of cohesion of the graphitic fault zone can be observed, with friction angle similar to the altered rock sample. The average angle of repose of the graphitic material was measured to be 41.5°.
Table 3  General strength parameters of dry fault material, (Golder Associates, 1993)

<table>
<thead>
<tr>
<th>Material</th>
<th>Cp (kPa)</th>
<th>φp (°)</th>
<th>Cr (kPa)</th>
<th>φr (°)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Graphite Altered Rock Sample</td>
<td>55.0</td>
<td>32.5</td>
<td>0.0</td>
<td>21.0</td>
</tr>
<tr>
<td>Graphite Fault-Zone Material</td>
<td>0.0</td>
<td>30.7</td>
<td>0.0</td>
<td>19.4</td>
</tr>
</tbody>
</table>

Point load testing was performed on a block of graphitic altered rock retrieved from a mineralized sector. The results indicate that the rock behaves anisotropically as point load testing carried out on the largest surface area of the graphitic block (perpendicular to the compression points of the apparatus) gave an Is(50) of 5.45MPa, while the tests in a vertical position gave an Is(50) of 0.97MPa. The estimated equivalent uniaxial compressive strength (UCS) are to be 131MPa and 23MPa, respectively. However these values appear to be on the high end of the laboratory compression range for mineralized core. This may be attributable to the irregularity of the sample tested under Point Load or the low sampling rate.

No UCS testing has been performed on the graphitic fault-zone material, as core is generally broken to cobblestones size fragments, thus rendering the sample unusable. Consequently the UCS is obtained by qualitative strength testing (Marinos and Hoek, 2001), the graphitic fault material is estimated to correspond to an equivalent UCS set between 1 and 5 MPa (i.e. highly weathered or altered rock, shale).

1.3   Mining method

Early production in the Casa Berardi Mine came primarily from mechanized cut and fill (MC&F), vertical retreat mining (VRM) and longhole open stoping (LOS). The mining lifts would reach up to 50m in height. Approximately 10m high sill pillars were left between MC&F stopes. Therefore the upper retreat mining (URM) method was used to extract portions of these pillars. Current practice at the mine favours the longhole mining method with production drill holes being 75mm (3'') in diameter on a burden-spacing pattern of 1,8m x 1,8m. Stope geometry is restricted to 20m in height and 15m in length (along the vein strike). The span of the stope can vary between 5m to 15m and is determined from the thickness of the vein. Longitudinal retreat mining is done primarily on veins averaging five metres or less in thickness (span less then 5m). When the vein is thicker, transverse primary/secondary LOS is applied with draw/drill points being developed every 15m centre to centre leaving a 10.5m pillar between the developments and limiting the stope span to 15m. If the span is greater (vein is wider than 15m), an A-B stope sequence is added. The graphitic faults strike approximately east-west. They often occur on either the foot or hanging wall of the sill development in longitudinal stopes or on the foot wall (north wall) for the transverse stopes. To minimise the potential damage of a fault unravelling, the mining front is limited to no more than three lifts (i.e. mining is systematically done under a sill mat). At the present moment, only the West Mine is in operation with 4 mining fronts spanning between 170m to 1000m below ground surface.

2   Case history

In the 1990’s, the CBM suffered three major events characterized by gravity-induced vertical “running” of the fault zone to the ground surface. The fault material exhibits little bulking on failure, and the collapses can run large distances within the rock mass and overburden – in excess of 150m. The overburden varies on average between 20m to 60m, consisting primarily of organic material, silt argillite/clay and glacial till capping the rock mass. Below is a summary of the events leading to the crown pillar failures as well as an examination of the distinct mechanics that led to them.
2.1 Major fault unravelling events

2.1.1 Description of the April 17th event (West Mine), 1992

While mining a 30m high uphole stope (SW200-03 Bloc; 27 1900t), with an incline of 20° to the north, 200m below ground level, major instability was suspected as an unusual amount of waste rock was being mucked out - three months after the initial production blast which was in August 1991. In November 1991, production subsequently continued in the adjacent bloc (SW200-04), but was stopped one month later as abnormal amounts of waste rock was also being recorded in the draw point. An investigation drill holes champagne was launched in January 1992. A void was intercepted above the SW200-03 bloc, with the base being 20m above the theoretical stope back and the void’s apex was within 30m to 50m of the overburden contact (local overburden thickness is about 40m). Production was immediately stopped and the potential material inflow was rapidly isolated by 6 temporary fill fences (cemented rockfill with at least 8m in thickness). Surface subsidence occurred on April 17th, it was reported that a 22m diameter crater was observed on the surface which resulted in the partial collapse of a water basin retaining wall. No water or material inflow to the mine workings was reported. It is suspected to have filled the void within the mining blocs SW200-03/04 (estimated volume of 13 304m³). Five (5) hydrostatic barricades were constructed in front of the temporary fences and the subsidence was back filled with sand and rock material. A permanent vertical pillar was placed 50m within the area of the estimated pipe. The fundamental cause of unravelling was attributed to a structural contact in the initial bloc, between a structural fault (strike 0° and dip 60°) and two graphitic auxiliary faults (striking 90°; dipping 65° and 80°, respectfully). The auxiliary faults were characterised by graphitic alteration and graphitic gouge material. As many access points per stope were used (waste rock was being discarded), the pull void was great enough to ultimately generate a bloc caving mechanism which ultimately connected the two stopes.

2.1.2 Description of the April 25th event (East Mine), 1992

Two incidences of progressive chimneying along the CBF from underground stopes up to the ground surface have been documented by Golder Associates (November, 1992). The collapses emanated from two longhole stopes developed over the top of the 150-4800 mechanized cut and fill stope front, located 100 m below ground surface. These stopes were the 100-340-4800 longhole stope (~25m open span along the fault) and the 100-350-4802 uppers stope (~25-30m open span along the fault). It is estimated that narrow cave zones progressively developed vertically from the open span of the longhole stopes by gravitational ravelling through about 60m of fault gouge material to the overburden (local thickness is about 30m). Both of these stopes were mining directly above cut and fill stopes that had exposure of the CBF, and had previously experienced ground control problems. The stopes were not filled immediately upon mining, leading to eventual unravelling of the fault gouge material. It is assumed that once the pipe reached the saturated soil, it could easily progress to the ground surface. On April 25th 1992, a first inrush of about 15,500m³ of saturated silts and clays entered the mine above the 100-340-4802 stope. Overburden silt and clay materials flowed into the breached stopes, eventually reaching the ramp before arresting at about the 150m Level with about 1m of mud on the ramp floor. A main crater was identified on surface, with a smaller crater situated to the west. The later was linked to the 100-340-4800 stope which had recorded major roof instability on fault contact, observed in October 1990. The exact date of its formation is unknown. One month after the initial inrush, the main pipe filled with till and silt finally collapsed (second inrush of about 15,500m³ of material), choking off further inflow of solid into the mine. This time, no additional material infiltration was recorded inside the main workings, as temporary fences were already constructed. Ground surface inspection have identified the growth of the main crater (from 60m to 70m in diameter). The west crater stayed relatively intact (15m in diameter). A drainage ditch was excavated around the crater area and pumps installed to remove accumulated surface water. An 8" diamond drill hole was drilled from surface to the 100m level and concrete was poured into the drifts that were connected directly to the cave area. The craters have been filled with rock to stabilize further growth. Within one year, the mine successfully isolated all possible access points with the caved sector by constructing twenty-three (23) hydrostatic barricades and numerous fill fences.
2.1.3 Description of the January 29th event (East Mine), 1997:

After the previous events in the East Mine, a rock pillar was established between the 250m and 285m Levels mining front in order to safely resume mining below the collapsed sector, without the risk of remobilizing further unconsolidated sand fill or any other mobile material present. Inside this pillar, it was decided to cut longitudinally through the fault and replace the graphitic material by an artificial concrete sill pillar (Mat Pillar). The intent was to stop/slow down any possible unravelling above this mat. A standard drift was constructed at the 275m level, leaving a skin of rock of 2.4m south from the CBF. Two subsequent perpendicular excavations (2.4mW x 4.0mH x 2.4mL) were excavated and the fault was exposed on the second cut. A fill fence was constructed and concrete was pumped. Subsequent cuts were done with a spacing of 4.8m centre-to-centre between cuts, leaving rock in-between. The rock was eventually excavated, and the void was poured with concrete as well. The final dimension of the artificial sill mat was 156m in length with interconnected concrete blocks (2.8mW x 4.0mH x4.8mL). The Mat Pillar ultimately failed (on January 29th, 1997) due to unravelling of the fault bellow the supporting pillar of the mat. The unravelling was initiated by the successive remobilization of unconsolidated backfill from inferior stopes (initiated by a 7 000m³ void along the graphitic fault). The exposed CBF fault quickly ascended to the 250m Level, remobilization 13,000m³ of unconsolidated sand fill from the superior cut and fill stopes. This material filled the void bellow and gradually migrating to surface. The following day, the small crater transitioned in size from 15m to 45m in diameter. A final hydrostatic bulkhead was created on the 300m Level in order to condemn the East Mine and allow further mining of the West Mine.

2.2 Chimney disintegration mechanism

The driving mechanism that led to these failures is a progressive chimney disintegration. Three types of possible chimney disintegration mechanism can be created (Brady and Brown, 2004). It is believed that two of them are the driving components in the failures that occurred at the CBM. The first mechanism is attributed to weathered or weak rock, with failure initiating on the stope roof or hanging wall, stabilizing in an arch like shape as bulking caved material provides self-support. This mechanism is systematically observed in all the graphitic fault material failures, either being a gravity induced failure due to the removal of a bottom support or a structural failure/weakening when exceeding the fault’s hydraulic radius. Furthermore, if subsequent void is created/generated, the cave progression resumes until a new equilibrium is attained. It is also suspected that the fault gouge material is characterized by a very low bulking factor during unravelling (Golder Associates, October 1992). In contrast with hard rock, the void generated within a graphitic instability appears to progress instantly (thus rapidly forming a chimney), without apparent loss of material at the initial contact point. Unravelled graphitic fault-zone material appears to have been pulverized, and appear as a flaky soil-like material.

The second mechanism is the result of unravelling of a discontinuous rock mass, as the rock itself may be characterized by strong UCS, the cohesion on the discontinuities create a plain of weakness as material is unable to self-support its own weight (Bétournay et al, 2003). It is well recorded through geological core logging that the rock mass is more fractured next to the graphitic faults. This ultimately weakens the rock mass cohesion as joints are altered with graphitic materials (acts as a lubricant). Field observations have shown that fault unravelling is usually always associated with some type of rock mass failure (primarily on the hanging wall due to stress relief), as the stabilizing arch is never exclusively located within the graphitic material. Thus the chimney pipe has to be large enough to mobilize the inflow material.

In order for these mechanisms to develop to the extent encountered at the CBM, sufficient void needed to be present beneath the caving fronts for continuing the chimney propagation. The void was either generated through drawing caved material or through the consolidation of uncremented backfill (creating potentially alternate caves along the CBF) ultimately interconnecting into a major conduit when the cave progressed to surface.
3 Evolution of ground support in graphitic fault zones

After Aurizon acquired the CBM in 1997, significant exploration and development was performed in the West Mine, with production reinitiated in 2006. During the development and the subsequent 9 years of production, excavation and ground support procedures were developed for stable mining through and along the CBF and other auxiliary faults. These procedures have been modified and improved as experience has been gained in mining over the years. These practices are strictly adhered to, due to the unforgiving nature of the fault material, and have been successful in control of unravelling along the faults. In fact, no major fault unravelling leading to surface collapse incidences have occurred at Casa Berardi since the operations were taken over by Aurizon Mines (followed by Hecla-Quebec). Below is a short summary of different trials in stabilizing the fault zone material and the final specifications currently employed at the mine.

3.1 Initial documented trials with supporting the fault

When exposing the CBF, some locations of the opening touching the fault may deteriorate almost immediately. Hence, different ground support strategies/techniques were employed in order to maximize the stability when driving through the fault zone material. The first strategy employed was to leave a thin skin of rock between the fault and the mining stope (between 1.5m and 4m). However, only partial success was obtained with 4m skin, which represented the thickness of a drift span. The second strategy was to directly bolt the graphitic material as standard drifts (standard ground support friction bars with mesh). Some success was obtained in controlling the unravelling when driving perpendicular through the fault (limited to the faults thickness). However, this method worked less for longitudinal development as support is installed on longer drifts along the fault strike. Another trial was performed using high pressure grouting within the fault gouge in order to create a pre-supported, cohesive material. The results have shown that the fault in general is not porous or fractured enough to allow the migration of infilling material. Due to operational delays and potentially exposing workers to an unravelling material, manual bolting was rapidly discarded. Finally, when shotcrete was introduced in the mine, it was discovered that the key to developing through/along the CBF was confining the graphitic material as quickly as possible after excavation.

3.2 Ground support specification for mining through graphitic faults

Procedures for mining through and along faults are as follows (Figure 4):

**Exploration** - Exploration diamond drilling is used to locate faults in advance of mining through them. The fault locations are added to standard mine geologic maps and taken into account in the stope planning.

**Drifting Through Faults** - When the drift is within approximately 2m of the fault, and until the drift has passed approximately 2m beyond the fault, the following practice is used:

- Engineering, geology and production planning meeting for design of every stope where special issues such as the condition or control of the faults can be discussed and alteration to plan made.
- Shorten blast rounds to 2m length.
- Use an arched-back profile to promote roof stability.
- Use spiling in advance of the face from drift shoulder to shoulder to prevent start of ravelling. Spiling (typically steel hollow pipe, Ø 1-¼") is nominally spaced at 0.4m and are 4.25m in length, which results in 1.5m of spiling bridging in advance of the face.
- Spraying a coat of nominal 15cm of fibre shotcrete around the entire drift surface and covering the spiling.
- Observe shotcrete for cracking and apply additional shotcrete as necessary.
Stoping Along Faults – Both longitudinal and transverse stopes are extracted in which the Casa Berardi or
other fault are typically found along one exposed wall of the stope. In these cases:

- Engineering, geology and production planning meeting for design of every stope where special
  issues such as the condition or control of the faults can be discussed and alteration to plan made.
- Drift through fault in development using procedure above.
- Control open stope span along fault:
  - Longitudinal stopes - no more than 6m width by 15m along strike by 20m in height (15mx20m
    exposure on the fault).
  - Transverse stopes – no more than 15m wide by 20m in height exposure on the fault.
- Perform a cavity survey of the open stope void and include digitally on mine plans.
- Fill stope with cemented rockfill or paste fill as quickly as possible after stope is mucked. The start
  of filling after mucking varies by experience within the zone as the susceptibility of the fault to
  ravelling varies by zone. Typically the stope is filled within 2 weeks of mining.
- Observe the stope during mucking to determine if any ravelling risk occurs so that adjustment to
  plans can be made if necessary to arrest the failure.

4 Operational challenges

Throughout the years, many operational challenges in developing through graphitic material have arisen at
the CBM. Listed below are the most potent ones, which the authors believe are of practical use to the
reader.

4.1 Respecting support specifications

When the fault thickness is important (≥4m), multiple round advancement needs to be undertaken. This
has shown to be a challenge due to the advancing face front converging to a concave shape. When the
blasted graphitic material is extracted by means of a LHD, it is regularly observed that a concave face can
easily be obtained if the operator does not take into consideration the friability of the material. Hence, the
subsequent round drilling will project and amplify the concave front, and unravelling will most likely
happen with blasting. This was the case with the loss of 118-890-360 draw point after 4 advances of 2m.
The event analysis showed that the spiling was installed too far from the mining front, leaving less than
0.25m of pre-support in the fault material. It was also determined that the physical "anchoring" limit is
1.00m. To remedy this situation, increased awareness amongst the working force was done. Operators
were asked to scrape minimal amount of graphitic material in the walls, to install the spiling as close to the
development front as possible, and to square the face when drilling the subsequent round. Coupled with
increased supervision, the fault drifting specifications were left unchanged and no further major events
were observed.

4.2 Unravelling failures

Unravelling of the graphitic material can be initiated by two types of failures: (i) a roof pillar failure (e.g.
developing through the fault); and (ii) a critical span failure (e.g. production induced exposure of the fault).

In the first case, when developing perpendicular to the fault strike, a 2m pillar span is created with a
hydraulic radius (HR) of 0.70m and 0.75m for a 4m or 6m drift span, respectively. The supporting vertical
walls of the drift present minimal sloughing. The graphitic fault is a soil type or weak rock material type,
hence the spiling support adds extra rigidity to the roof pillar beam created. Hence, augmenting the
natural stand up time of the graphitic material. Through observations, it was concluded that unanticipated
roof pillar failure occurs when the spiling specifications (mainly nominal support spacing) are not respected.
Figure 4  Procedures for mining through and along faults; (a) specification longitudinal and cross section view; (b) face drilling after completing the spiling installation with jumbo boom; (c) result of a successful development through the CBF fault, with secondary support added to anticipate production (friction set and mesh strap bands).

It is observed that this support is time dependent, as creeping/sloughing of the graphitic material can occur between the steel bars or in the corners of the upper portion of the walls. Fault material must be confined by shotcrete within the practical stand up time of 24h used by the mine. In order to allow blasting through the fault, the shotcrete machinery must be declared functional and be positioned within one mining level of reach. Experience has shown that longer pre-support installation is possible, but not recommended as there is a loss in drilling precision thus compromising the spacing between supports.

When developing parallel to the fault strike, major considerations need to be taken in evaluating the extraction ratio of the fault material. As with the current ground support specification, depending on the thickness, the fault is usually entirely driven into with a maximum permissible graphite/rock ratio of 33% or less in the face front. Full development within the fault is not recommended at the CBM, as the longitudinal drifts are usually 50m and more in length. Consequently, mining techniques can be adapted by
transitioning from longitudinal to transverse stoping (change in development strategies). Alternatively, a skin of rock can also be left in place between the drift and the fault, but experience has shown that the skin’s thickness must be at the least equivalent to the span of the drift otherwise dilution/unravelling of the fault will occur.

In the second case, stope production systematically exposes/mobilizes graphitic fault material. Unravelling of the fault occurs progressively as the ore is being pulled out from the draw point, ultimately the bulk of the fault can be mined out (Figure 5). The loss of confinement in the upper walls of the stope are common, but unravelling does not occur as long as the fault is confined in the critical sectors of the stope (primarily in the roof). Secondary support (split sets and welded mesh) is added to the shotcrete for this purpose, and to reduce blast induced shearing. Monitoring have shown that the critical span for exposing/undercutting the fault material is 30m and based on past events, failure can occur rapidly beyond this point. Stope stand up time is crucial to not incur additional instability. It is influenced by the total production volume and the thickness of the fault material. Stope extraction always presents risks, since stope stability is never absolutely guaranteed by analytical/numerical analysis. Hence, to ensure viability of a production sector, mining is always done under a sill mat (used as an intervention level) to stop the propagation of a potential unravelling or chimney. Therefore, a maximum of 3 vertical stope lifts are sequences. Once the stope is mined out, backfill is rapidly undertaken. Stopes along the fault have higher priority for backfilling, with preference to cemented paste fill. Continuous surveillance of critical stope is undertaken by the mine’s technical service personnel. If uncontrolled instability arise, backfilling will commence immediately. The current critical production front is the 118 zone.

![Figure 5](image-url)  
*Figure 5*  Transverse stope along the fault CBF, final CMS profile in response to the theoretical production outline; (a) section view looking north, unravelling can be seen in upper shoulders; (b) longitudinal view looking west, complete degradation of the CBF fault.
5 Conclusions

Casa Berardi Mine has successfully developed a ground support procedure for mining through graphitic fault material, present in the CBF and auxiliary faults. The graphitic material is characterised by soil type properties: low swelling, cohesion less material and reduced stand up time. Based on the case history events and through operational observations, the mine has learned to attenuate the potential of inducing a major chimney disintegration failure through drift development and mining along/next to the fault. Hence, the initial unravelling failure is controlled by pre-supporting and confining the fault with shotcrete and by efficient time minimization strategies, to extract the mineral and rapidly backfilling the production stope. Production mining, with the presence of thick graphitic fault, is always done under a sill mat located at a maximum of 3 lifts above the mining front. Continuous monitoring of the fault during this period is crucial.

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Ground support modelling involving large ground deformation: Simulation of field observations – Part 1

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Abstract

The Kristineberg mine has a long history of large ground deformation which consequently incites ground control problems for the mine. Over the years the mine has developed various mining techniques, backfilling and ground support procedures to manage this problem. In general the ground control problems at the mine are highly influenced by the wall rock geology. The wall rock, that is the footwall and hangingwall, comprise of highly altered chlorite schist, which are internally referred to as talc-schist. They very often occur as seams with thickness barely ranging from 0.1 m to as wide as 3.0 m. Coupled with high ground stresses the talc squeezes and slides into the stope if undercut by the excavation, or either bends or bulges inwards when exposed but not undercut depending on the loading direction. The deformation magnitudes have often been reported to be in the order of 0.2 to 0.5 m and seldom up to 1.0 m. Conventional rock support system, consisting of fibre re-enforced shotcrete and rebar rock bolts, has regularly failed under these conditions. As part of Ground Support Research Initiative at Luleå University of Technology a monitoring program was designed to measure ground deformation and the response of the ground support system. Numerical modelling was conducted to capture the responses as observed during monitoring. The numerical models revealed all the typical mechanisms of instability that have been conceptualized through observations and earlier studies. Talc obviously was the most influential lithology that controlled the deformation characteristics of the stope and ultimately on the rock support system. Combinations of bending, bulging, shearing and tensile mechanisms induced a complex loading pattern on the rock support system. Often the rock bolts, for example, would experience all of these mechanisms at once or during different stages of the excavation rounds as a cut is developed.

1 Introduction

The Kristineberg mine, owned by Boliden Mineral AB, has a long history of large ground deformation which causes stability problems for the mine. Over the years the mine has developed various mining techniques, backfilling and ground support procedures to manage the problems associated with those problems. In general the ground control problems at the mine are highly influenced by the wall rock geology. The wall rock, that is the footwall and hangingwall, comprise of highly altered chlorite schist, which are internally referred to as talc-schist. They often occur as seams with finite thickness barely ranging from 0.1 m to as wide as 3.0 m. Coupled with high ground stresses the talc squeezes and slides into the stope if undercut or exposed depending on the loading direction. The deformation magnitudes have often been reported to be in the order of 0.2 m to 0.5 m and seldom up to 1.0 m. The conventional rock support system, consisting of fibre re-inforced shotcrete and rebar rock bolts, has regularly failed under these conditions.

To study the rock support response to large ground deformation the Kristineberg mine therefore provided an ideal test site. Hence, a field investigation was conducted at the mine in the J-orebody in a stope located at a depth of 1200 m. Monitoring was conducted when Cut #4 of stope J10-3 was mined. The monitoring involved; convergence measurements, total station surveys, borehole photogrammetry, damage mapping, and instrumented bolt measurements. Two systems of rock support were utilized in the investigation with the objective of evaluating their performances. The rock support systems were: (i) shotcrete + rebar and (ii)
shotcrete + D-bolt. They were installed in alternating 5 m long rounds as the cut advanced. Bolting was regularly spaced in a 1.0 m by 1.0 m pattern.

To construct the numerical models the geology of the stope was extracted from the drill-hole data and stope face mapping. First, the numerical models were simulated without ground support to study the deformation and failure characteristics of the rock mass around the stope. Next, the rock supports were installed and the response of the rock support to the observed ground deformations was analysed.

The numerical models revealed all the typical mechanisms of instability that have been conceptualized through observations and earlier studies (e.g. Krauland et al, 2001; Board et al, 1991). The talc schist obviously was the most influential lithology that controlled the deformation characteristics of the stope and ultimately the rock support system performance. Combinations of bending, bulging, shearing and tensile mechanisms induced a complex loading pattern on the rock support system. Often the rock bolts for example, would experience all of these mechanisms simultaneously or during different stages of the excavation rounds as the cut is developed.

It is difficult to make precise conclusions regarding the performance of the two rock bolt types utilized in the test stope. This is due largely to two reasons: (i) the instrumented bolt measurements were quite unsuccessful due to many of them failing prematurely and (ii) the deformation magnitudes experienced in the test stope were much smaller than expected and thus inducing less strain on the rock bolts. Nevertheless, the convergence measurements on the rock bolt heads did indicate to some extent that, the D-bolt being ductile, appears to relax with increasing deformation, while the rebar bolt being stiff does not. This observation is made at least after the stope has advanced more than 30 m from the point of measurement, by which time the stope convergence has settled.

2 Background of test site

2.1 Kristineberg mine

Boliden Minerals’ Kristineberg Mine is one of the deepest mines in Sweden and it is located in the municipality of Lycksele, Västerbotten county, in northern Sweden. Mining at Kristineberg presently takes place between 850 and 1320 m below the ground surface. The ore bodies are polymetallic and contain zinc, copper, lead, gold and silver. They are mined using a mechanized overhand cut-and-fill mining method for narrow orebodies and drift-and-fill for wide orebodies (greater than 8 m). Stopes are typically 150 m along the strike and are developed from the center and mined in each direction by breasting. The cuts are typically 5 to 6 m high and 6 m wide and are excavated in 5 m rounds by drilling and blasting. One stope is approximately 50 to 60 m high.

2.2 Ground control problems

The primary ground control problems at the Kristineberg mine are closely related to the geological conditions around the stopes. The presence of the weak chlorite quartzite and talc-schist in both sidewalls and the weak interfaces between the rock units along the hangingwall and footwall contacts results in failures in the sidewalls in a pattern which is fairly common in the mine. The failure initiates within the weak talc-schist, followed by slip along the contact if it is undercut by stope opening.

As the stope is excavated, the relatively hard ore in the roof is subjected to increased stresses, thus forcing the ore to punch into the weak sidewalls and then drag downwards. In the process it induces shearing in the footwall, roof-parallel fractures and typically bending failure in the hangingwall. The behaviour is illustrated by Figure 1.
3 Field monitoring

3.1 Investigation site description

Figure 2 shows the plan of the site of field investigation. Stope J10-3 shown in the figure is located at a depth of 1195 m in the J-ore zone. The field investigations were conducted in Cut #4 of Stope J10-3. Investigations in Cut #5 of J10-3 and Cut #1 in Stope J10-4 were part of a different project and therefore are not reported in this paper.

The vertical sections through Y825, Y850 and Y875 are shown in Figure 3. R1 to R10 represent the breasting rounds. The rebar bolt and D-bolt were alternated within these rounds in order to study their respective performances. To conduct numerical simulations vertical sections of the stope were also extracted as shown in Figure 3 and local geology as observed in the bore holes through sections Y825, Y850 and Y875 were added to the sections to complete the models for simulations.

Figure 2 Stope J10-3 Cut #4 where field investigations were performed.
Figure 3  Stope profiles through sections Y825, Y875 and Y850. Monitoring was conducted in Cut #4.

3.2 Monitoring

The monitoring involved; convergence measurements, total station surveys, bore-hole photogrammetry, damage mapping, and instrumented bolt measurements. Convergence measurements involved both tape and bore extensometers. The instrumentation pattern and relevant monitoring methods are illustrated in Figure 4. Sections S3:4 and S3:6 are high density monitoring sections where all monitoring systems stated above were utilized and were necessary to study the behaviour of the ground support.

3.2 Field monitoring results

A summary of the field results are presented in this section to facilitate the numerical simulations presented later. Figure 5 shows the maximum convergences observed from tape extensometer readings. These convergences depend on the local geology in these sections as well as the distance from the stope face. There is some evidence from S1 to S4 where the D-bolt appears to relax, allowing more deformation as the cut is reaching its maximum convergence. Figure 6 shows the borehole images over the 42 days of monitoring and Figure 6 and 7 show the convergence of the footwall between sections S5 and S6 where damage mapping showed cracking of shotcrete and corresponds to Figure 5c where the footwall shearing is quite significant.
Figure 4 Monitoring plan. S1 to S10 represent the instrumented sections. Sections S3:4 and S6:7 are high density monitoring sections (after Perez & Nordlund, 2014).

Figure 5 Maximum HW-FW convergence with alternate rebar and D-bolt supported rounds.
Figure 6 (a) HW convergence in section S1 and (b) HW borehole closure in section 1. The colour dots represent days when observations were made.

Figure 6 (a) FW convergence in section S2 and (b) FW borehole closure in section 2. The colour dots represent days when observations were made.

Figure 7 (a) Shear movement in the FW observed between sections S5 and S6 where shotcrete cracking were also observed. (b) FW borehole closure between S5 and S6.
2.3 Discussions on field monitoring results

It is difficult to make precise conclusions regarding the performance of the two rock bolt types utilized in the test stope. This is due to two reasons: (i) the instrumented bolt measurements were quite unsuccessful due to many of them failing prematurely and (ii) the deformation magnitudes experienced in the test stope were much smaller than expected and thus inducing less strain on the rock bolts. Nevertheless, the convergence measurements on the bolt heads does indicate to some extend that, the D-bolt, being ductile, appears to relax with increasing deformation, while the rebar bolt being stiff does not. This observation is made at least after the cut has advanced more than 30 m from the point of measurement, by which time the stope convergence has settled.

Surface failure mapping by visually assessing and recording damages to shotcrete and failure of rock bolts were carried out during the monitoring period. Although, majority of the rockbolts remained intact, the shotcrete showed cracking as the faced advanced. Cracks were immediately evident when the face advanced 5 to 6 rounds ahead.

3 Numerical simulations

3.1 Unsupported models

Numerical analyses were performed for profiles Y825, Y850 and Y875 (Figure 4) with their respective geology obtained from the boreholes that intersected them. As they are unsupported models no ground supports were applied. The objective was to study the ground deformation behaviour without support.

Tables 1 and 2 show the inputs used in the numerical models. The in-situ stresses are those from hydraulic fracturing measurements reported by Stephansson (1993) as:

\[
\sigma_v = 0.027Z, \quad \sigma_H = 2.8 + 0.04Z, \quad \sigma_h = 2.2 + 0.024Z
\]

where; \(\sigma_v\) is the vertical stress, \(z\) is the depth, \(\sigma_H\) is the maximum principal stress and is parallel to the orebody, and \(\sigma_h\) is the intermediate principal stress and is perpendicular to the orebody.

<table>
<thead>
<tr>
<th>Material</th>
<th>Youngs modulus, (E) (MPa)</th>
<th>Poissons ratio, (\nu)</th>
<th>Rockmass compressive strength, (c_{cm}) (MPa)</th>
<th>Cohesion, (c) (MPa)</th>
<th>Friction, (\phi) (°)</th>
<th>Tension, (\sigma_t) (MPa)</th>
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</thead>
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<td>23.2</td>
<td>6.7</td>
<td>30</td>
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<tr>
<td>Chlorite-quartzite</td>
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<td>0.7</td>
<td>0</td>
<td>36</td>
<td>0.04</td>
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* Backfill parameters after Knutsson, 1981.

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<th>Shear stiffness, (k_s) (GPa/m)</th>
<th>Cohesion, (c) (MPa)</th>
<th>Friction, (\phi) (°)</th>
<th>Tension, (\sigma_t) (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>6000</td>
<td>60</td>
<td>0</td>
<td>5</td>
<td>0</td>
</tr>
</tbody>
</table>
3.1 Results from unsupported models

The ground deformation behaviour for the unsupported stopes through profiles Y825, Y850 and Y875 are shown in Figures 8 to 10, respectively. In profiles Y825 and Y850 the talc occurs both in the HW and FW. The undercutting of talc in the HW resulted in significant ground deformation in the HW. In the FW the deformation is small since the talc seams are located further behind the FW surface. Total station measurement in Section 6 (S6), inserted in Figure 8, confirms the deformation behaviour observed from the numerical simulation for profile Y850. No total station measurements were available for profile Y825. Camera images through the borehole located in the FW, between S5 and S6 (i.e. between R5 and R6), show significant shearing of the FW talc (see Figure 7).

In the profile Y875 the talc occurs in the FW and also through the middle of the orebody. The talc seam in the middle of the orebody is fully undercut by the stope, while the seam in the FW is partially undercut. The deformation of the talc in profile Y875 occurs in the form of shearing, sliding and squeezing. The total station measurement of Section 3 (S3), inserted in Figure 10, is the closest total station measurement to profile Y875, since none was available for Section 1 (S1). The inserted total station measurements for S3 shows similar deformation pattern as observed numerically in profile Y875. In fact, the talc seam that is located on the FW of Y875 appears to extend to S2 and S3. This is clearly evident from the fact that, the borehole camera images in the FW at S2 show significant shearing within the talc (see Figure 7).

The numerical simulations of the unsupported stopes show a number of mechanisms involved in deformation of the talc including; bending, shearing and dilation. These mechanisms are important to know as they will ultimately affect the response of the rock support elements.

Figure 8  Ground displacement behaviour around profile Y825 in Cut #4. Large deformations occur in the HW where the talc-schist seam has been undercut.

Figure 9  Ground displacement behaviour around profile Y850 in Cut #4. Large deformations occur in the HW where the talc seam is closest to the open stope. Inserted on the right is the total station profile through Section 6 located in the vicinity of Y850.
3.2 Supported models

Profiles Y825, Y850 and Y875 were also simulated with rock supports installed. Profile Y875 was supported with D-bolt and shotcrete, while profiles Y850 and Y875 were supported with shotcrete and rebar to conform to the actual installation. The rock bolts are 2.7 m long and regularly spaced 1.0 m apart. Table 3 gives the properties of the rock bolts. The D-bolt, considered a high strength-ductile rock bolt, has two components; the non-deformable anchors and deformable shanks. Like the rebar the D-bolt yields at 0.2% strain, however, while the rebar is assumed to fail at yield (which may not be true in practice), the D-bolt is reported to strain plastically up to 14% before failing (see Li, 2010). In the numerical simulation the D-bolt is modelled in two parts; the non-deformable anchors are assigned very high strength and modulus, while the shanks are assigned the reported strength and modulus of the D-bolt. However, to make the D-bolt strain plastically a residual strength equal to the yield strength is assigned. Table 4 shows the fibre re-enforced shotcrete parameters.

<table>
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<th>Rebar</th>
<th>D-bolt</th>
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<td>Anchors</td>
<td>Anchors</td>
</tr>
<tr>
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</tr>
<tr>
<td>Residual tensile strength</td>
<td>100 000</td>
</tr>
</tbody>
</table>

3.2 Ground reaction curve

A ground reaction curve (shown in Figure 11) was created for the stope and calibrated against monitoring data in order to assist in determining when the rock support is to be installed in the models. In the ground reaction curve the critical limit is reached when the rock mass modulus is reduced to about 25% of the original value, resulting in inward displacement of about 60 mm. However, for the numerical modelling the 20% limit for modulus reduction is chosen. With this limit a convergence of 75 mm was obtained, which is when compared to the convergence measurement data, that approximately 50% of the convergence would have occurred before the ground support elements (shotcrete and rock bolts) were installed.
Figure 11  Ground reaction curve for the stope. The critical limit is reached when the modulus is reduced to 25%. For modelling 20% limit for modulus reduction is chosen.

3.2 Results of the supported models

Figures 12 to 13 show the results from the supported stope models. The yielding of the rock bolts correspond to the areas where the talc layers are actively deforming. In profile Y825 segments of the rock bolts in the HW have yielded in tension, corresponding to the HW talc that has been undercut and is deforming into the stope. Similarly in profile Y850 the rock bolts also yielded in tension in the HW, where the talc is actively deforming by bending. In Profile Y875 the rock bolts that intersected the talc seams in the roof and FW have yielded in tension. The yielding are consistent with the mechanisms that drive the deformation of talc around the stope. For example, the rock bolts installed in the talc seam in the roof of Cut #4 in profile Y875 are yielding in tension since the sliding of the talc down the middle of the stope induces tensile stress due to the downward drag and high lateral stresses. This phenomenon is known to drive the back parallel cracks at the Kristineberg mine, leading to the church dome shaped roofs.

Profile Y875 particularly shows significant yielding of the shotcrete liner all around the profile. This profile is in fact located in the area where large shear movements were observed in the FW via the borehole camera (see Figure 7). In the HW borehole camera images showed closure of the borehole as the HW converges into the stope (see Figure 6)

Figure 12  Cut #4, Profile Y825 is supported with D-bolt. Segments of D-bolt have yield in tension mainly in HW side, where the talc has been undercut and deforming into the stope. Yielded shotcrete segments are shaded red
4 Discussions

It was not possible to develop geological models that accurately represent the geology around the stopes because of incomplete and missing data. However, by comparing the drillhole geological data and the face and roof mapping that was carried out in cuts #3 and #4, as well combining geological knowledge it was possible to develop geological models of the wall rock through profiles Y825, Y850 and Y875. The borehole camera snapshots and failure mapping of cut #4 also indicated that, the deformation mechanisms observed were consistent with the interpreted geology.

Numerical modelling based on the “real” geology was necessary as it would then be possible to compare field observations against results from numerical simulations, as well as identify geology and mechanism that control ground deformation around stopes. Furthermore, it provided the basis for model calibration.

The models with real geology clearly showed the talc to be the most influential lithology that controlled deformation around the stope. Three main mechanisms associate with talc were observed; shearing, dilation and bending. Bending typically occurs if the talc seam is thin. In this case it is forced to separate from the contact by compressive force from the ore in the roof and the lateral stresses. Dilation occurs with thick talc seams, where they squeeze and “belly” into the stope by dilating. Shearing is coupled with dilation.
Since the talc-schist is very weak it is punched by the ore into the roof immediately above the stope. The ore then drags downwards under its own weight and in the process forces the talc-schist to slip along the clay filled contacts.

The segments of rock bolts and shotcrete apparently yielded in areas where the talc was actively deforming. It was not possible to evaluate the difference between the performance of the D-bolt and rebar, since the deformations in the test stope were smaller than anticipated. The majority of the strain gauged rock bolts failed and data was unreliable, making it difficult to assess the performance of the rock bolts.

It is also possible that an entire length of a rock bolt can be located inside the weak talc, if it is sufficiently thicker than the bolt length or parallel to the bolt application as in in profile Y875. In this instance the bolt will be straining with the talc, in which case the bolt may not be useful in anchoring the talc to the competent host rock.

The deformation of the stope is also a function of face advance. This can be seen from the fact that deformation continued to increase as the face advanced, with furthest rounds showing the largest deformation compared to rounds closest to face. Furthermore, shotcrete cracking was not seen in the early stages of the excavation until the final rounds were excavated. It is also estimated that at around 50% of the deformation has already occurred prior to installation of monitoring instruments. Displacements tracked from the unsupported numerical models show displacements to be almost twice that observed in the test stope, which of course were measured in supported ground conditions.

5 Conclusion

The interaction between ground support and rock mass during large ground deformations are complex and require an understanding of the mechanisms that drive these deformations. As learnt from the Kristineberg experience these mechanisms are significantly influenced by the characteristics of the wall rocks. At the Kristineberg mine a combination of shearing, bending, bulging and direct tensile and compressive loading were observed, thus subjecting the rock support elements to complex mechanisms often occurring simultaneously. The influence of talc-schist on the overall ground deformation behaviour was clearly evident.

Acknowledgement

The authors wish to acknowledge LTU, LKAB and Boliden Mineral AB colleagues involved in the ground support research and financing provided by Boliden, LKAB, VINNOVA and Centre of Applied Mining and Metallurgy (CAMM) at Luleå University of Technology.

References

Ground support modelling involving large ground deformation: Simulation of conceptual cases – Part 2

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E.Nordlund, Luleå University of Technology, Sweden

Abstract

As a continuation of Part 1 of ground support modelling involving large ground deformation by Saiang and Nordlund (2016) this paper presents the conceptual models and results from the typically observed cases throughout the Kristineberg mine. The Part 1 of the paper focused primarily on results from measurements carried out at the J-orebody, whereas the Kristineberg mine consists of many ore bodies or lenses within the VMS (volcanic massive sulphides). Part 2 therefore presents some of the typically observed rock mass behaviours throughout the mine. The mine geology is very complex for each of the orebody due to the multiple phases of wall rock alterations and various geological processes that occurred throughout the history of the deposit. This history also included both local and regional events of folding, faulting and shearing. Despite the notable differences in the local geology around the different orebodies, there is nevertheless, a general trend that the stability of the stopes throughout the mine are in principal controlled by the altered wall rocks, the presence of other lithologies capable of inhibiting deformation and the geometry of the stopes themselves as demonstrated in Part 1.

1 Introduction

The Kristineberg deposit is hosted in a zone of chloritized and sericitized quartz and feldspar-rich acidic volcanics. The ore zone strikes E-W and the dip varies between 45° and 70° S, and plunging approximately 45° SW. The host rock is a schistose seritic quartzite which ranges from a strong quartzitic rock to a weak highly altered material close to the proximity of the orebody. Immediately bounding the orebody are highly weathered chloritic talc-schist, which vary in thickness from 0 to 3 m or more. The material is very weak, friable and greasy with schistocity. The contacts between the chloritic talc-schist and the ore body or the neighbouring competent rock are often planar with talc coatings with low friction. The contact may also be a shear zone of thickness up to 1 m or greater, which is composed of altered material including schist and sericitic quartzites and occasionally bands of pyrite are found within these zones. The local geology and their spatial characteristics have significant impact on the deformations characteristics of the stope walls and consequently the reaction of the ground support to these deformation patterns.

Saiang and Nordlund (2016) focused on results from measurements carried out the J-orebody. However, the Kristineberg deposit consists of roughly 8 orebodies or lenses, with varying geometrical characteristics and wall rock geology (see Figure 1). These orebodies are exploited either by cut-and-fill or drift-and-fill mining methods depending on the rock mass and orebody characteristics.

Despite the notable differences in the local geology around the different orebodies, there is nevertheless, a general trend that the stability of the stopes throughout the mine are in principal controlled by the altered wall rocks, the presence of other lithologies capable of inhibiting deformation and the geometry of the stopes themselves. Simulations have been performed for the typically observed cases which are believed to be representative of the mine and the results are presented herein.
Figure 1. The orebodies of the Kristineberg deposit (courtesy of Boliden Mineral AB).

2 Typically observed cases

A number of observed cases have been presented by Krauland et al (2001), Board & Rosengren (1992), Board et al (1991) and in internal publications by Boliden Mineral AB. The mechanisms illustrated by Board & Rosengren are shown in Figure 2. Scenario B in Figure 2 has commonly resulted in offsetting of stopes often sighted at the Kristineberg mine. It is generally observed that the spatial characteristics of the talc-schist, particularly locality and thickness, significantly influence failure and deformation characteristics of the stopes. The degree of alteration decreases from the hangingwall to the footwall. This means the talc-schist layers in the footwall are weaker than those in the hangingwall. Furthermore, they are often thicker in the footwall (FW) than they are in the hangingwall (HW).

The talc layers sometimes make up the contact between the ore and host rock, other times they are isolated with the host rock or can be even found in the middle of the orebody. Their thickness can vary from 0 to 3 m. Bands of interlayered pyrite are also seen within the talc-schist zones. Figure 3 demonstrates these basic scenarios.

3 Conceptual cases

In consultation with the rock mechanics engineers at Kristineberg mine the typically observed scenarios were expanded from the basic cases shown in Figure 3. These expanded cases are shown in Tables 1, 2 and 3. The cases in Table 3 are geometrical scenarios where the cuts in a stope are offset from each other, due to offsetting of the orebody by faulting, shearing or folding. The offsetting of the cuts are thought to have notable effect on the deformation characteristics of the stopes. Even though the conceptual models are simple they do realistically represent some of the common characteristics of the wall rock at Kristineberg mine, particularly the talc and spatial characteristics.
Figure 2  (A) Failure patterns and mechanisms associated with weak contact zones at the Kristineberg mine (B) A complex geology, where a shear zone intersects the orebody leading to a large zone of chlorite schist.

The geology for the conceptual models have been simplified into four main units: (i) the orebody, (ii) host rock, (iii) talc-schist zones and their clay filled contacts and (iv) the pyrite zone. It is believed that interlayered zones of pyrite reduce inhibit free shearing and dilation of talc zones within the stope walls.

Ground support is also applied in the conceptual models to observe how they respond to the different conceptual cases. That is, both unsupported and supported stope simulations were conducted. The installation of the rock bolts and shotcrete follow the same pattern as applied in (Saiang & Nordlund, 2016).

The inputs for the rock mass, ground support elements and backfill are same those used in (Saiang & Nordlund, 2016). The ground support reaction curve developed in Part 1 is also applied to the conceptual models here for support installation.

Figure 3  (a) thin talc-schist band in the HW and thick band in the FW, (b) Talc-schist in the footwall is farther from the ore contact, (c) the talc-schist in the middle of ore and (d) interlayered pyrite bands associated with talc.
Table 1  Conceptual scenarios for talc (red colour fill) in the FW with relatively thin layers in HW: (A1-A3) thin layer with varying distance, (B1-B3) medium layer with varying distance, (C1-C3) thick layer with varying distance, (D1-D3) with thin layer through the middle of the orebody.

<table>
<thead>
<tr>
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<td>C3</td>
<td>D1</td>
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</tbody>
</table>

Table 2  Conceptual scenarios for talc (red colour fill) and pyrite seams (blue fill) in the FW, with thin seams of talc in the HW

| E1 | E2 | E3 | F1 | F2 |

Table 3  Conceptual scenarios for faulting and shearing: (G) Fault partially displaces the orebody and (H) faulting fully displaces the orebody

| G1 | G2 |

4  Results from conceptual cases

Table 4 shows the summary of the results from the numerical simulations of the conceptual cases. In the first column of the table are ground deformation behaviours without ground support for the various cases. In the second column are the ground support reactions from models simulated with ground support elements installed. In the third column are interpretations and conclusions drawn from behaviours observed.
Table 4  Conceptual models, numerical results and conceptual descriptions of the rock mass behaviour around the stope. The unsupported ground responses are represented by maximum shear strain contours and displacement vectors.

<table>
<thead>
<tr>
<th>Ground response - unsupported</th>
<th>Rock support response</th>
<th>Interpretations</th>
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<tr>
<td><strong>Case A1</strong></td>
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<td></td>
</tr>
</tbody>
</table>
| ![Image](image1)              | ![Image](image2)       | *The ore punches into the weak thin talc layers on either side of stope shoulders and forces the talc to break and separate from the host rock.*  
*Then the ore drags downward due to its weight and the high lateral stresses, while inducing shear slip along the contact. Meanwhile the broken section of the talc exposed in the stope FW and HW become free standing talc beams which bend and fail by crumbling.*  
*Back-parallel cracks develop in the roof due to forces resisting downward drag of the ore and the high lateral compressive stresses.*  
*The ground support (rock bolts and shotcrete) mostly yield in the roof where significant sliding and straining occur. In the FW the yielding of the rock bolts are mostly associated with shearing of the talc.* |
| **Case A2**                   |                        |                 |
| ![Image](image3)              | ![Image](image4)       | *The FW corderite separating the stope and talc seam appears to restrict free straining of the talc seam into the stope. Thus straining appears to concentrate in the roof on the HW side.*  
*In the roof the ore punches into the weak talc seam in the HW. Because the seam is relatively thin it is broken as in Case A1. The ore is dragged downwards on the HW side, inducing sliding along the contacts of talc seam and forcing it to squeeze into the stope starting from at the shoulder.*  
*The shearing of the talc seam in the FW causes the FW to bend on the lower half of the wall.*  
*Ground support mostly yields in the roof and shoulders. In the FW the yielding of the rock bolts are associated with shearing of the talc.* |
| **Case A3**                   |                        |                 |
| ![Image](image5)              | ![Image](image6)       | *The ground response behavior in the roof is similar to that in Case A2.*  
*In the FW the 3 m corderite footwall rock separating the talc seam from the open stope appears to have significant impact on restricting the straining of the talc towards the open stope. However, the down shearing of the talc seam is not restricted, resulting in a circular shear movement near the toe of the stope and upward heaving of the backfill.*  
*Yielding of the ground support mostly occurs in the roof and shoulders due to sliding of the ore largely from the hanging slide.* |
Case B1

- The behaviour is similar to that in Case A1. However, the deformation in the footwall is greater than in Case A1 due to thicker talc seam.
- Notably the ore in the roof of the ore punches into the weak talc seam in the FW, forcing it to bulge immediately under the FW shoulder. Because the seam is relatively thick it is not broken. The ore is dragged downwards on both sides, inducing sliding along the contacts of the talc seam and forcing it to squeeze into the stope.
- The downward drag of the ore is notable on the FW side.
- The ground support yields in the roof because of the downward drag of the ore.
- In the FW the ground support is yielding due to shearing of the talc and the bulging of talc as ore compresses the talc after punching through the FW near the shoulder.

Case B2

- Similar behavior as in Case A2. Slightly greater movement in the footwall than in Case A2 due to thicker talc seam.
- The cordierite FW rock separating the talc seam from the open stope obviously restricts free straining of the talc towards the open stope.
- Hence, the downward drag of the ore mostly occurs towards the HW side where sliding occurs mostly along the thin talc band on the HW.
- The ground support yields in the roof and in the shoulder of the HW, which corresponds to the movement described above.
- In the FW the rock bolts yield in tension inside the talc zone due to bending that is resulting from the downward shearing of the talc.
- The circular shearing at the toe of the stope causes the FW to bulge near the toe of the stope, causing the shotcrete to yield.

Case B3

- Similar behavior as in Case A3 and B2.
- The farther the talc seam is to the open stope the less it deforms directly into the stope. However, the downward shearing of the talc forms a circular failure surface at the toe of the stope. Floor heaving of the backfill is obvious.
- Yielding of the ground support is largely occurring in the roof due to downward sliding of ore.
- The farther talc seam is from the stope the less the impact it has on the ground support.
### Case C1

- The behaviour is similar to that in Cases A1 and B1 except the deformation is quite large in the footwall because of the much larger talc seam.
- The ground support yields significantly in the roof and FW.

### Case C2

- The behaviour is similar to that in Cases A2 and B2 except the deformation is larger in the footwall because of the thicker talc seam.
- The 1 m thick cordierite FW rock notably prevents the ore from punching into the talc as in Case C1. So there is no swelling of the talc directly under the FW shoulder.
- However, significant downward shearing of the talc leads to bulging near the toe of the FW.
- The rock support yields notably in the roof of the HW side where the ore is free to slide but not on the FW side.
- The downward shearing of the talc causes the ground support to yield mostly on the lower half of the FW.

### Case C3

- The behaviour is similar to that in Cases A3 and A3 except the deformation is larger because of the much thicker talc seam.
- However, the movements towards the open stope are significantly restricted by the thicker zone of competent cordierite FW rock.
- Downward shearing of the talc expresses itself at the toe of the FW resulting in heaving at the FW toe and backfilled floor.
- The ground support mostly yield in tension in the roof on the HW side where the ore is free to slide and rotate (as it is restricted on the FW side).
The talc in the roof causes the ore to dislocate, separate and slide on the FW side and in the process it attempts to drag the ore on the HW side with it.

The ground behaviour is similar to that in Case D1, however the competent FW cordierite in the FW prevents the ore from punching into the FW talc.

But, the downward shearing of the talc continues and attempts to express itself at the toe of the stope with a circular shear characteristics.

The ground support yields in the roof and in the footwall inside the talc.

However, on the lower half of the stope the rock bolts experience more yielding than in the upper half of the stope.

The ground behaviour is similar to that in Case D2, however the competent FW cordierite in the FW prevents the ore from punching into the FW talc.

The 3 m thick FW cordierite significantly restricts the deformation of talc.

But, the downward shearing of the talc continues in the FW.

The ground support yields in the roof and in the footwall inside the talc.

However, on the lower half of the stope the rock bolts experience more yielding than in the upper half of the stope. This is because the shearing is more pronounced near the toe.
Case E1

- The behavior is similar to that in Case B1, except that the pyrite seam appears to significantly resist free shearing of the talc into the open stope.
- Nevertheless, the ore still punches into the talc seams (both in FW and HW) and compresses the talc seams as it drags downwards.
- The ground support yields in the roof due to downward movement of the ore.
- In the FW the ground supports yield due to shearing of the FW talc.

Case E2

- The behavior is very much similar to that in Case E1.
- For the pyrite to be effective in arresting free deformation of the talc in the stope it must be present on the inner side. While the talc is outside relative stope wall.
- The ground support yields in the roof close to the FW shoulders. On the lower half of the FW the rock bolts yield due shearing of the talc.

Case E3

- The behavior is very much similar to that in Case E2. The talc seam is relatively thick compared to the thin pyrite seam. Hence the behavior is also similar to that in Case C1.
- The ground support yields in the roof and the FW shoulder. On the lower half of the FW the rock bolts yield due shearing of the talc.
In the FW the shearing of round supports – This may also imply that the step.

The HW talc seam deforms inwards in the form of bending. Shearing is minimal.

Horizontal deformation occurs in the footwall, but shearing is apparently non-existent.

In the roof the ore punches into the talc seams on the sides (HW and FW). The downward drag of the orebody is noticeable in the HW but in the FW.

This may also imply that the step-wise mining of the stopes may restrict large shear deformation in the FW. The disturbance to the continuity of the ore and talc seams, caused by faulting and folding, seem to affect the stope deformation behavior significantly.

Rock bolts yield, notably in the roof and HW, where significant straining. occurs

In the FW the shearing of talc causes the shotcrete to yield. The rock bolts also yield within the talc zone.

### Case F1

- The behavior is very much similar to the in Case C1.
- The effectiveness of the pyrite seam restricting free straining of talc is not noticed unless it is located on the inside of the talc or between the talc and the FW.
- The ground supports yields in the roof and the FW as in Cases C1 and E3

![Case F1](image)

### Case F2

- The pyrite seam effectively cuts off the ore from punching into the talc.
- Downward shearing of the talc continues behind the pyrite seam.
- The ground support yields notably in the HW roof and possibly less in the FW roof.
- In the FW the rock bolts yield mostly as a result of downwards shearing of the talc which causes the rock bolts to bend and yield in tension
- As in the previous cases the downward shearing expresses itself as circular shear near the toe of the stope.

![Case F2](image)

### Case G1

- The HW talc seam deforms inwards in the form of bending. Shearing is minimal.
- Horizontal deformation occurs in the footwall, but shearing is apparently non-existent.
- In the roof the ore punches into the talc seams on the sides (HW and FW). The downward drag of the orebody is noticeable in the HW but in the FW.
- This may also imply that the step-wise mining of the stopes may restrict large shear deformation in the FW. The disturbance to the continuity of the ore and talc seams, caused by faulting and folding, seem to affect the stope deformation behavior significantly.
- Rock bolts yield, notably in the roof and HW, where significant straining. occurs
- In the FW the shearing of talc causes the shotcrete to yield. The rock bolts also yield within the talc zone.

![Case G1](image)
5 Discussions

The effects of talc on the deformation of the stope are affected by several factors:

- **Thickness of the talc seams:** Thin bands of talc generally respond by bending, after being separated along the contact. Talc seams that immediately surround the ore are easily broken in the roof as the competent ore punches into the soft talc. Once broken the weight of the ore is applied directly on to the standing talc band causing it to flex and fail by crumbling. Thicker seams of talc, which are typically found in the footwall, generally squeeze and dilate into the stope. This is caused by a combination of the high lateral stresses and the downward compression caused by the weight of the ore. “Bellying” of the FW also seems to be associated the thick talc seams. Wedges of cracks will appear in the middle of the FW height as a result. At the base of the FW and under the backfill, wedge shaped circular failure may occur. The backfill will heave if the shear failure occurs under the backfill.

- **Location of the talc seams:** The farther the talc seams are from the stope, the effect diminishes. The competent rock mass separating the stope and talc seam behaves like a rock pillar. The nearer the talc is to the stope, the competent rock mass in between behaves like a slender pillar. If the talc seam is thick, the impact on the slender rock pillar is significant, as it is forced to bend into the stope.

- **Occurrence of pyrite seams with talc:** Pyrite seams appear to resist shearing of the talc, due to its high frictional resistance. This observation is consistent with the field observations made in Cut # 4 where lenses of pyrite were seen in the camera boreholes in some of the sections.

- **Undercutting of talc seams:** The mechanisms that drive deformation and failure are also affected by whether the seams are undercut or not. If the seams are not undercut by the stope then bending is the primary mechanism. If the seams are undercut then shearing is the primary mechanism.

- **Talc through middle of the ore:** If the talc seam is in the middle of the ore and is undercut, it is forced to extrude by the high lateral stresses.

Other phenomena have also been observed:

- **Development of back parallel cracks in the roof:** The parallel to sub parallel cracks appear when the ore is dragged downwards, under its weight, with the talc contact acting as the sliding surface. The downward drag of the ore can be visualized as acting on “fix-free” boundary. The movement is fixed on the boundary where the talc is absent; while it is free to move downward along the boundary where the talc is present. Therefore movement is first initiated from the wall where the
talc is present and because it is fixed on other wall a rotational moment is created, leading to church dome shaped back parallel cracks. A rock bolt would be subjected to shear, bending and rotational moment under this scenario. If talc is present on both sides (HW and FW), the ore tends to slide into the stope under gravity, as well as being forced by the high lateral stresses. Back parallel cracks develop and propagate into the back until a state of equilibrium is reached. A rock bolt would be in tension under this condition.

- **Effect of faulting, folding and shearing:** Faulting, folding and shearing results in irregular orebody geometry. This results in cuts that offset each other, both up dip and along the strike. The offsetting tends to cause less ground deformation for the subsequent cuts. This scenario was clearly observed in the experimental stope, where cuts are offset and is likely to be one of the main reasons for the low deformation magnitudes observed in Cut #4. From the ground control point of view the offsetting of cuts could be an advantage.

Since the maximum principal stress is known to be perpendicular to the orebody then bending will be main the deformation mode if shearing does not occur. This is clearly seen in the conceptual models where the displacement vectors are perpendicular to stope walls where the talc is either absent or thin. Although the conceptual models show instabilities in FW, HW and the roof, the instabilities in the roof are significant with respect to rock fall and immediate safety. Obviously the stability of the walls is important for stabilizing the roof, since instability in the walls would provide a pathway for the entire instability leading up to the roof. Most of the fallouts would have occurred at the same time as the blasting. This is evident from the profiles in the test area where fallouts have occurred during blasting near the HW and FW shoulders and other areas talc has been undercut.

### 6 Conclusion

The conceptual cases which realistically represent the observations at Kristineberg mine show many facets of the ground deformation characteristics; bending, shearing, dilation and bulging, and direction tension and compression. These mechanisms occur either simultaneously or sequentially depending on the advancement of the cut, characteristics of the wall rocks and the stope geometry. Consequently the ground support elements are subjected to all these mechanisms either simultaneously or sequentially too. The spatial characteristics of the highly altered talc-schist has a major influence in the stope deformation characteristics, while the presence of pyrite is noted to reduce to deformation to some extent by resisting the shearing of the altered talc-schist.

### Acknowledgement

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Ground support practice at Glencore’s nickel rim south mine – with a link to seismic monitoring data

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Abstract

Glencore’s Nickel Rim South Mine, located in Sudbury, Ontario, Canada, is a primary/secondary blasthole operation with ore zones lying between 1,100 and 1,700m below surface. As per other relatively deep primary/secondary mines (Mercier-Langevin 2010), the secondary stopes are mined in a yielded rockmass state. Ground support requirements are variable and must cater for high stress primary stopes in abutment or sill pillar regions (burst prone support) as well as relaxed blocks of ground (secondary stopes) where open rockmass fractures allow mine process water inflow that often causes significant corrosion issues. Dynamic design and static/corrosion resistant design can be essential elements of installed support depending on the stope access. Furthermore, abutment regions have a yield zone width that is on the order of the typical panel width (12.5 to 15m) depending on the orebody thickness. These abutment regions are in close proximity to high rockmass stress that can generate large magnitude seismic events. As with many mines, the loading history and conditions can vary considerably. Seismic monitoring is the mine’s premier tool to track the actual rockmass response to mining. Yielded zones have a reasonably clear seismic signature/pattern over time that is visible in the collected data. The ability to accurately locate small magnitude seismic events can help define and delineate different stages of rockmass failure. This helps focus ground support requirements (e.g. need for rehabilitation, dynamic versus gravity loading areas), and can impact positioning of both equipment and manpower underground.

A description of the different ground support systems (past and present) and their performance is given. Examples from the seismic monitoring program showing yielded, yielding, and “intact” areas are also provided, as well as their implications for ground support design. Accurate seismic event locations are critical for rockmass response interpretation. Improvement in seismic location accuracy by using 3-dimensional velocity models is demonstrated.

1 Introduction

Glencore’s Nickel Rim South Mine (NRSM) is located along the eastern flank of the Sudbury Igneous Complex between the Greater Sudbury Airport and Lake Wanapitei. Both nickel and copper ore zones are extracted with significant precious metal credits, especially in the copper zones. The mining method is sublevel open stoping with cemented hydraulic tailings backfill. The local stope sequencing varies, but is generally a primary/secondary transverse method, where the secondary stopes (pillars) are extracted with “just-in-time” (delayed) development in a yielded rockmass state (Jalbout and Simser 2014). The “just-in-time” development avoids the secondary access being subjected to stress changes and blast vibrations from adjacent primary stope panel mining. The rockmass through which the secondary access will be driven is allowed to deform and fracture prior to the tunnel being driven. At the peripheries of the orebody, where zones narrow, either pillarless end slicing or longitudinal mining is utilized. A key element to the method is to ensure the stope pillars are slender (width to height ratio < 2/3) so they do not store excessive energy. End slicing along the abutments can facilitate similar behaviour, with a zone of yielded ground offering a zone of low energy storage (lower rockburst risk) where stope accesses are driven, depending on the timing of the development. A long section and cross section of the NRSM is shown in Figure 1.
Seismic array

The seismic network is supplied by ESG Solutions¹ and consists of a mix of 15 triaxial geophones (15 Hz) and 36 uniaxial accelerometers for an average sensor spacing of approximately 150m in the main mine area. A blended signal processing logic that combines data from both accelerometers and geophones is used to maintain an accurate source parameter range from approximately moment magnitude -2 to +2. 4.5 Hz geophones placed in the far field are used for large event magnitude estimates as well as for correlation to the Sudbury Regional Network (Hudyma and Beneteau 2010).

First 9 months of 2015 near stoping areas

Figure 2 Typical Gutenberg-Richter relationship for near stoping seismicity at Nickel Rim South Mine. The linearity of the plot is an indication of the reliable calculation of seismic moment

The seismic event location accuracy varies but typically ranges from 5 to 15m for most of the data shown in Figure 2. Although this is considered good for most mine-wide seismic systems, both Glencore and ESG are

¹ Engineering Seismology Group Canada Inc., a Spectris Company, Kingston, Ontario, Canada.
continually evaluating procedures that can improve the location accuracy of seismic events. More accurate seismic event locations lead to a better understanding of rock mechanics issues. Stope panel widths are from 12.5m to 15m at NRSM. The inference of rockmass response to mining from seismic data is often hampered by location errors of similar dimensions to the volume of interest (e.g. stope pillar). Ongoing work utilizing 3D velocity models (Collins and Toya 2015) that take into account differing lithology and mining voids is significantly improving source location accuracy. An example is discussed in section 6.

3 Seismic response to mining

The geology of the NRSM is a mixture of hard igneous host rocks with both nickel rich and copper rich semi-massive to massive sulphides as ore zones. Both zones are relatively steeply dipping and are bulk mined using blasthole methods. The hard nature of the host rocks (unconfined compressive strengths (UCS) >150 MPa) and high rockmass ratings (GSI or RMR >60) results in microseismic activity being generated when highly loaded (during the rockmass failure process). Figure 3a shows the geological complexity on a section of the 1660 level. The relatively high sensor density at the NRSM provides the ability to routinely record seismic events in the moment magnitude -2 range, providing a reasonable record of the rockmass failure process. Visible damage from any individual small seismic event is rare. Typically, damage is observed only for seismic events above magnitude 0.5, depending on excavation proximity to the source and other factors such as local site response. However, the tracking of the rockmass failure process by the seismic system provides useful insight about how the rockmass is responding to mining over time (Simser et al. 2015). For example, Figure 3b shows that clear seismic gaps exist in the yielded secondary stope pillars (white gaps between the blue stope cavity surveys) when looking at seismic data recorded in 2015, even while mining these stopes. The overall centre-out mining sequence is also very apparent with abutments and sill pillar regions showing higher seismic activity rates. Information from the seismic system confirms that one of the major goals of the overall mine design is being achieved; stress is being pushed to the outer edges of the mine and underground worker exposure to high stress ground conditions is being dramatically reduced.

Figure 3  (a) Detailed geological mapping from the 1660 meter (below surface) sublevel. The grid spacing is 25m. This copper zone is a mixture of erratically distributed copper veins (red) in a granitic host rock (felsic gneiss in orange, Sudbury breccia in blue) with occasional brecciated dyke fragments (brown). (b) Long section of the mine with near-stopping seismicity plotted for 2015 (38,211 seismic events total). Seismicity is being “pushed” to the outer edges of mining

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2 in the Nickel Rim context where rockmass quality is usually good in all the major rock units
The source mechanics of the seismic activity varies, and often cannot be spatially separated by specific geological units due to the complexity. For example, the relatively soft copper veins (30 to 120 MPa UCS) may deform and load the adjacent stiffer/stronger host rock. The host rock emits the seismic activity but is in immediate contact with the vein. Wider veins may be up to 5 to 10m thick but in most cases mineralized veins are less than 5m. Similarly, faults can deform with sudden movement that emit recordable seismic waves, or more slowly via aseismic creep. It is often observed that recordable seismic activity is increased between a fault and a stope, with the fault channelling stresses into an effectively smaller solid window (Jalbout and Simser 2014). Regardless of the magnitude or mechanism, the seismic event is a record of inelastic damage in the rockmass at the location recorded.

4 Ground support systems employed at the nickel rim south mine

The bulk of the underground project development for the NRSM was completed between 2007 and 2009 using in-cycle shotcrete and resin grouted rebar. The first mine stoping began in May 2009. The ground control support system worked well initially but signs of the shotcrete peeling due to its brittle nature became apparent after several years of stoping. Evidence of this is shown in Figure 4a.

![Figure 4a](image1.png) ![Figure 4b](image2.png)

Figure 4 (a) Open stope brow area with original stope access shotcrete peeling due to fracturing around the opening, seismicity and blast vibrations. (b) Similar brow area but with weld mesh (screen) over shotcrete to accommodate higher deformation

During a mine project phase, the driving economic force is early ramp-up of ore tonnage. The high quality rockmass and low overall extraction was very amenable to fibre reinforced shotcrete and resin grouted rebar. Inevitably, near-stopping areas will have increasing deformation; both pseudo-static (rockmass bulking due to stress induced fracturing) and dynamic (blasting vibration and seismic activity). The ground control and mining teams adjusted to these changing conditions by installing weld mesh (screen) over shotcrete (Figure 4b) or in some areas, by installing bolts and weld mesh without shotcrete.

The wet shotcrete process utilized a borehole delivery system which operated reliably from 2009 to 2014. However, by 2014, wear and eventual borehole obstructions led to the temporary loss of the shotcrete delivery system. More reliance on bolt and mesh without shotcrete was necessary until alternative shotcrete delivery methods could be established.

Any support system is a balance between operational ease, cost, and performance. Evolution of the support elements is also a natural progression that follows the extraction of an orebody where overall abutment stresses will increase with time. Furthermore, faults become more mobile with increased mining as they are pierced by an increased number of stopes. The number of larger seismic events also increases with increasing extraction. This is shown in the summary of the evolution of ground support systems at NRSM shown in Table 1.
Table 1  Summary of the evolution of ground support systems at the mine. Includes rounded production figures and the number of large seismic events per year. The case history section and Figure 5 show some examples of in situ support performance

<table>
<thead>
<tr>
<th>Year</th>
<th>Dominant Support System</th>
<th>Enhanced Support</th>
<th>Supplementary Support</th>
<th># Events &gt; Mom. Mag. 1.0</th>
<th>Ore Extracted (t)</th>
</tr>
</thead>
<tbody>
<tr>
<td>2007 to 2009</td>
<td>50mm fibre reinforced shotcrete with 1.2x1.2m pattern of 2.4m resin rebar</td>
<td>Occasional 00 gauge mesh straps, pillar noses, structural/blocky area</td>
<td>Cement grouted cables in intersections, 3.5m super Swellex bolts for stope brows</td>
<td>0</td>
<td>550,000</td>
</tr>
<tr>
<td>2010</td>
<td>50mm fibre reinforced shotcrete with 1.2x1.2m pattern of 2.4m resin rebar</td>
<td>Occasional 00 gauge mesh straps, pillar noses, structural/blocky area</td>
<td>Introduction of pumpable resin grouting for cablebolts</td>
<td>1</td>
<td>1,000,000</td>
</tr>
<tr>
<td>2011</td>
<td>50mm fibre reinforced shotcrete with 1.2x1.2m pattern of 2.4m resin rebar</td>
<td>Introduction of yielding bolts and straps or mesh strap 30x30cm plates, mesh over shotcrete</td>
<td>Resin grouted cables, 3.5m super Swellex bolts for brows</td>
<td>4</td>
<td>1,300,000</td>
</tr>
<tr>
<td>2012</td>
<td>50mm fibre reinforced shotcrete with 1.2x1.2m pattern of 2.4m resin rebar</td>
<td>Wide use of yielding bolts and mesh as outer layer near stoping areas</td>
<td>Resin grouted cables, 3.5m super Swellex bolts for brows</td>
<td>16</td>
<td>1,300,000</td>
</tr>
<tr>
<td>2013</td>
<td>Mix of rebar and mesh or fibre reinforced shotcrete and rebar</td>
<td>&quot;Pre&quot;-hab of select strike drives in anticipation of mining induced stresses (yielding bolts and mesh over original shotcrete)</td>
<td>Mining under backfill (shotcrete and Swellex bolting) for sill pillar overcuts, coated Swellex in some areas</td>
<td>18</td>
<td>1,300,000</td>
</tr>
<tr>
<td>2014</td>
<td>Shotcrete borehole problems, more rebar/mesh than in cycle shotcrete</td>
<td>Wide use of yielding bolts and mesh as outer layer near stoping areas</td>
<td>Wider use of No.4 gauge mesh versus No.6 gauge mesh</td>
<td>24</td>
<td>1,300,000</td>
</tr>
<tr>
<td>2015</td>
<td>Mix of in-cycle shotcrete or rebar mesh, recognition that corrosion issues in secondary stopes will be a long term problem despite No.4 gauge galvanized mesh</td>
<td>For actively spalling ground – in-cycle shotcrete to reduce strainburst risk to mechanized bolter operator followed by yielding bolts and mesh over top</td>
<td>More filling of stope accesses with cemented backfill to mitigate wall undercutting and confine draw point pillars</td>
<td>17</td>
<td>1,300,000</td>
</tr>
</tbody>
</table>
The ground support systems used at NRSM cater to three main conditions:

- **“Normal”** ground conditions for moderately blocky/fractured ground far from stoping induced stresses.
- Stressed ground near mining or areas where mining induced stresses will affect the ground at a later mining stage (rockburst resistant support).
- Yielded ground (secondary stope accesses) where gravity is the dominant driving force for ground failure but overall rockmass quality has been reduced by previous nearby mining.

There is a trade-off between operational simplicity (one size fits all) and cost. For example, the full enhanced support package shown in Table 1 consists of in-cycle shotcrete (50mm), followed by No.4 gauge mesh and resin grouted yielding bolts on a 1.2x1.2m pattern. The 2.4m long yielding bolts are often supplemented by cablebolting in intersections where spans are wider or in areas where adverse faulting/jointing is encountered. Applying this standard mine wide would be prohibitively expensive and unnecessary.

For normal ground conditions, empirical experience in the Sudbury mining district and the “1/3 span rule” for minimum length of bolts are used. Resin grouted rebar offer high capacity immediate support and good corrosion resistance. At NRSM, both reinforced shotcrete (50mm with synthetic fibres) or weld mesh (No.6 gauge, 4.9mm diameter strands, 100mm square aperture) have worked well as areal support in conjunction with the 22mm diameter, 2.4m long resin grouted rebar on a 1.2x1.2m pattern.

For stressed ground, bursting support has evolved in the Canadian Mining industry over the last 20 years (Simser 2016) and a variety of yielding bolts and other support elements are readily available. A combination of heavy mesh (5.7mm strands) and double plates (30x30cm 00 gauge mesh strap plates over 15x15cm domed plates - see Figure 5 for examples) with a 1.1x1.1m pattern of yielding bolts (Li 2010) is used. The double plating helps load transfer to the tendons and reduces the likelihood of the square edged steel plate guillotining the weld mesh sheet strands.

The secondary stope pillars are evaluated using information from the seismic system, as well as underground observations, and are almost always determined to be yielded ground in the vicinity of the primary stoping. These pillars are accessed “just-in-time” after the adjacent primary stopes have been mined. This avoids blast vibrations and stress changes (loading cycles on the tunnel) during access development. Minimal seismicity above moment magnitude -2 is experienced once the secondary access tunnel enters the yielded ground around the primary stoping. Gravity and corrosion then become the main factors to consider for support design. Resin grouted cablebolts and rebar are the main bolt types used. Occasional splitset style bolts are required in the walls if excessive cracking or backfill from an adjacent primary stope is encountered.

An artefact of the yielded rockmass is that mine process water often permeates through open fractures, especially given the nature of the hydraulic backfill that is poured at 72% solids. This water permeation has proven corrosive and even No.4 gauge galvanized mesh has not been sufficient for the full life cycle of the stope access tunnels where the overcut eventually becomes the next tier of mining levels’ undercut. Both shotcrete and mesh have been used for surface support, however shotcrete has proven superior due to its’ corrosion resistance and ability to reinforce weaknesses in the rockmass. Flexibility is allowed to choose from in-cycle fibrecrete and rebar (occasionally friction bolts are required in the walls due to poor ground obstructing resin cartridges) or spraying plain shotcrete over mesh if shotcrete resources are temporarily limited. The ground movements encountered are typically within the capacity of 50mm thick shotcrete with the main exception being wall undercutting by fanned blastholes (overcuts and undercuts usually driven at 5 to 6m wide for 12.5x15m wide panels).

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3 1/3 span = minimum bolt length
Figure 5  Different support systems used at Nickel Rim South Mine (a) Shotcrete spalling (b) Yielding bolts and mesh over shotcrete. Over time, the shotcrete became cracked and dilated but the deformable bolts and mesh installed over the shotcrete holds the system in place. (c) Secondary stope access with corrosion evident in the mesh but shotcrete still intact without visible cracking. (d) Corroded 00 gauge mesh plates (8.4mm diameter strands) and No.4 gauge galvanized mesh (5.7mm strands). Corrosion was localized around a sulphide vein. (e) Resin grouted cables exposed by breasting down the back of a large excavation showing excellent grouting quality
5 Case history – western abutment

In some areas at NRSM, end slicing (pillarless centre-out retreat sequence) is used to avoid leaving squat secondary pillars. End slicing is also used at the limits of the ore zone. The stope overcut shown in Figure 6a was created by mining under backfill which is in a low stress state. The top half of the overcut is driven through backfill while the lower portion is located in heavily fractured rock. The undercut access shown in Figure 6b traversed through a stressed abutment into yielded ground. The fractured ground tends to allow mine drill and backfill water to percolate through it. This water inflow is not seen in stressed ground that has not yet fractured (i.e. primary stope development access).

Figure 6 (a) Overcut under backfill, the top half of the opening is in fill and the bottom half of the tunnel is in fractured rock both having no significant strain energy and thus lower bursting risk. (b) Portion of the undercut in pre-fractured ground. A distinct characteristic is that fractured ground is porous allowing mine drill and backfill water to percolate through the fractures. Stressed/unfractured areas have no water inflow. Natural ground water is rare at Nickel Rim South Mine at these depths

Figures 7- 12 show the seismic response of mining in the lower west abutment of the mine at 1680m below surface during various phases of mining from October 2011 to December 2015. Figure 7 shows the general layout of the 1680 western abutment area and a sampling of the recorded seismicity during the 2015 calendar year.

Figure 7 (a) Area of interest in the lower west abutment of the mine viewed looking North. (b) Lower sublevel (undercut level) at 1680 meters below surface viewed looking Southeast. One year of seismic data is shown (2015) and the mining above has been removed from view for clarity
Figure 8a shows a plan view of development blasts recorded by the seismic system during the western crosscut development period. The seismic system parses out events into 200ms windows so each blast sequence is recorded, and appears, as several blasting events. The solid black line in the figure represents the actual survey centre line of the stope access tunnel. Note that the blast locations are systematically offset from the survey line by approximately 5m to the west. Simser et al. (2015) discuss the use of development blasting for seismic system calibration and seismic event location evaluation and attribute this shift in blasting locations to waveforms travelling through more heavily fracture rock (i.e. incorrect velocity), and/or raypaths travelling around mined out stopes. Both scenarios result in the seismic waves generated by blasts taking longer to reach the sensors than expected if a single velocity/straight-line raypath is assumed by the seismic system. The result is blast locations being calculated at further distances from the sensors than reality. Work to reduce this seismic event location offset bias includes the use of 3D velocity models that accounts for mining voids. This is discussed further in Section 6 of this paper.

Figure 8b shows a plan view of seismicity in the area of interest with moment magnitude > 0.0 (October 2011 to March 2015) and includes the Mag 1.2 April 22 event that provided an opportunity to evaluate local support performance. It is clear that when only seismic events above Mag 0.0 are considered, most of the seismic record of rock mass fracturing is not visible. Significant rockmass damage is occurring over time due to the cumulative effect of fracturing that generates only very small magnitude events. Information from these smaller seismic events contributes greatly to an enhanced understanding of the rockmass response to mining during the mining cycle.

The April 22 seismic event at 20:18:02 (Mag 1.2) shown in Figure 8b caused a small rockburst in the main strike drive approximately 66m from the calculated seismic event location. The damaged area is shown in Figure 9 and was located at a brecciated dyke fragment that stored strain energy locally. However, no damage was observed in the adjacent crosscut shown in Figure 6b, even though it was located only 50m from the same Mag 1.2 seismic event. This nearby crosscut was heavily supported and was in a yielded region with minimal locally stored energy. Localized rockburst damage can be triggered by the ground motion resulting from a larger seismic event, which tens of metres away from the source, has particle velocities only in the mm/s range. These ground motions can act on the rockmass by triggering a “coiled” spring load effect in highly stressed ground that can result in rock ejection in the m/s range (Ortlepp 1993).
Figure 9  April 22 lower wall burst where 50mm of fibre reinforced shotcrete was installed (below the lowest line of rebar). The darker coloured rock is a brecciated diabase dyke fragment (UCS >300 MPa). Figure 6b shows the crosscut which had no visible damage from the event.

Figures 10 to 12 step through the recorded seismicity in the western abutment area over different time increments. The area is in the lower abutment region of the mine and is relatively highly stressed. The highly stressed rockmass coupled with the relatively dense seismic sensor array of the ESG monitoring system results in a substantial amount of recorded seismicity in this part of the mine. Many very small seismic events are recorded with moment magnitude down to approximately -2.0.

Figure 10a shows seismic data from October 2011 when the first primary stope was mined on the 1680 level. The amount of seismicity recorded shows how the primary stoping leads to significant fracturing of the surrounding rockmass that will in turn, become future secondary stopes. Figure 10b shows seismic data from October 2011 to March 2015 that includes seismicity generated by several local stope extractions.

Figure 10  (a) Seismic response from first stope mined on the 1680m level (lowest sublevel in the mine). The seismic activity on either side of the primary stope (black arrows) indicates that the future secondary stopes end up in a fractured/yielded state over time. (b) 3.5 year period (October 2011 to March 2015) of recorded seismicity in the region is shown. This data includes seismicity generated by several local stope extractions (arbitrary cut off of selection volume). The location of damage from the April 22 Mag 1.2 seismic event (shown in Figure 9) is also indicated.
Figure 11a shows seismicity recorded from March to May 2015 when the western access crosscut was developed. Figure 11b shows data from June to September 12, 2015, corresponding to the time between development and actual stope extraction. Note the seismic gap during this time period, indicating that the rockmass had already been fractured in this volume and is no longer responding seismically to any increased loading.

Figure 11 (a) Seismicity from March to May 2015 when the western crosscut was developed. (b) Recorded seismicity between development and actual stope extraction (June to September 2015). Note the lack of seismicity recorded in the seismic gap zone indicated. This seismic gap is located in the same rockmass volume where recorded seismicity had been significant from October 2011 to March 2015 (Figure 10b).

Figure 12a shows data recorded on April 22 when the Mag 1.2 event occurred. Highlighted are the seismic event location and the location of observed damage. The damaged/burst area plots as a small seismic cluster with dimensions comparable to the brecciated dyke fragment in the area. The ground motion from the April 22 Mag 1.2 seismic event adversely affected the rockmass with locally stored energy where low deformation capacity support (shotcrete and rebar) was installed (Figure 9).

Figure 12 (a) Seismicity recorded on the day of the Mag. 1.2 April 22 seismic event. (b) Seismic response to mining the western abutment stope (28,000 tonnes) from September to December 2015.

Figure 12b shows the seismicity recorded from September 13 to December 2015 when mining the western abutment stope (28,000 tonnes). The seismic gap correlates to stress shadowed/yielded ground in the
abutment area. This is also where the remote stand for mucking the stope was located. The crosscut in the abutment (Figure 6b) had heavy gauge mesh and yielding bolts, and was in ground that had already yielded at the location closest to the seismic source. No significant local strain energy was stored in the rockmass and no damage was observed even though it was closer to the April 22 event. The conclusion is that no “coiled spring” situation existed in the heavily fractured crosscut.

6 3D velocity models for Improved seismic event location

More accurate seismic locations are necessary to infer more detailed rockmass behavioural changes due to mining. For example, the typical stope width at NRSM is 12.5 to 15m. Draw point pillars are in the 5 to 10m range. Stress/strain analogies (Simser et al. 2015) such as energy index (stress proxy) and cumulative apparent volume (strain proxy) are more meaningful when clear demarcations between yielded, yielding, and loading can be made. Otherwise, the volume selection of seismic data could contain mixed data from very different mechanical processes (geological structure controlled failure vs. intact rock fracturing for example). The incorrect selection of rockmass volumes, and associated seismicity for data analysis, could potentially obscure the end result of any type of higher order analysis.

Recent work by Collins and Toya (2015) have shown that 3D velocity models that can take into account both changes in lithology and mined out areas can greatly improve source location accuracy. Figure 13 shows how seismic event location accuracy can be improved with the use of a 3D velocity model. Figure 13a shows stope blasts located with a 1D constant velocity model. These blasts are known to have occurred when blasting the new stope as indicated. Most of the blasts, however, locate approximately one level above the stope. This realization provides a good indication that any seismic events located within the area could be mislocated by an equivalent distance. Figure 13b shows the significantly improved blast locations relocated with a 3D velocity model that includes both open and filled stopes as well as individual velocities for specific geological units at NRSM. The improved seismic source locations are critical if ground control decisions are expected to be made based on an accurate understanding of the rockmass conditions at the resolution desired at NRSM.

Figure 13 (a) 1D Constant velocity model showing a group of stoping blasts locating approximately one mining level above the actual known locations of the blasts. (b) Same blasts as in (a) relocated using a 3D velocity model that accounts for different velocities for major geological units and mine voids
At NRSM it was found that the mined out volumes have the biggest influence on location accuracy. Ray paths bend around backfilled or open stopes and travel through the rockmass. The ray path actually travels a further distance and takes longer to arrive than expected from source to sensor. A constant velocity model would interpret this extended raypath time as being caused by a source at greater distance from the sensor. The end result is that seismic event locations have an apparent distance offset further from the actual source location. The goal at NRSM for 2016 is to implement more sophisticated location algorithms such as 3D velocity models in quarterly time steps. Stopping and backfill processes will act as inputs to a re-evaluated 3D velocity model as mining continues.

7 Conclusions

The primary/secondary stoping method in the deep mining context creates two conditions; stressed ground, and relaxed/yielded ground. A one-size-fits-all ground support system is not effective from either a cost or a performance perspective. The secondary stope access tunnels are subject to flowing mine water that can cause corrosion of any steel support members. The pre-damaged rock responds well to in-cycle shotcrete where defects (joints, blast fractures, stress cracking, etc.) are strengthened. Deformation is typically still limited because the main driving forces are gravity and local blast vibrations. Fibre reinforced shotcrete or mesh reinforced shotcrete with rebar and back cables are the main support elements used.

As the overall extraction of the deposit increases, primary stope accesses are subject to high stress changes and in some cases may be developed into a rockmass with high stress conditions. In-cycle shotcrete in this context is too stiff for the life cycle deformation of the tunnel, but it does have several benefits. Shotcrete can lower the strainburst risk to the bolting operators by providing some confinement to the rockmass and, by default, delaying the bolting cycle which allows time for seismic activity to decline. Shotcrete also reduces rockmass dilation, thus promoting better load transfer to the outer shell of mesh and yielding bolts. Dilated material has a better opportunity to unravel out around the bolts, whereas the tighter rockmass provides more confinement around the bolt (most bolt types at least partly rely on friction to transfer load to the tendon). For cost reasons, areas that are driven under moderate stress conditions but are expected to be later subjected to higher stress may be driven without the first pass of shotcrete.

The seismic data can be used to infer the state of the rockmass as it changes with time. The data can help ascertain if a volume of rock is in the process of yielding or has yielded and will no longer generate significant seismicity. This in turn provides insight into ground support planning. The full rockburst resistant systems being used in Canadian Mines are successfully containing rockbursts, but they are still significantly more expensive to deploy than conventional support. Selective use, that can lead to cost savings, is only possible through a better understanding of rockmass behaviour. Tracking this behaviour given the complexity of many mine geological environments is an essential tool for the ground control engineer and mine management.

Seismic monitoring provides a real time 3D feedback loop to measure the rockmass response to mining. High resolution arrays with good location accuracy can provide significant detail on the rockmass response to mining. The use of 3D velocity models versus the more traditional constant velocity assumption for seismic source location algorithms will improve this analytical capability further. It is clear that as mining progresses and the raypaths become more convoluted (bending around stoped out areas), seismic event location accuracy can only degrade with time under the constant velocity assumption. The ability to update the 3D velocity model by including new mining voids into the model can help restrict this degradation and ensure the most accurate information is available to the ground control engineer at all times.

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References


Grouting and excavation support in Doha – Simple, but challenging

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Abstract

The Doha metro is an 85-km railway network in Qatar. There will be approximately 100 stations built for the entire Metro Network. Keller has executed anchoring for 13 stations and grouting works for 5-stations for the construction of three different lines of the Metro stations and cross passages. During the execution of works, many different geotechnical challenges have been covered.

For the anchor works, various anchor systems from Europe and Asia have been used as per client requirements. This has presented some challenges since a major part of these anchors are for a 10-years life spans and of removable type. During the anchor installation, various difficulties such as cavities and high water inflow of up to 300 l/s have been encountered and resolved.

Aside from the excavation support measures, the grouting works for ground-water control were carried out in 5 metro stations. Excavation pits with dimensions of up to 40m by 800m were treated using the tube a manchettes (TAM) method. Additionally, at a different station, Keller was requested to reduce the ground water inflow for a flooded station. This was successfully executed using a specially-adjusted grouting scheme during ongoing dewatering works. The client was then able to excavate further without any serious water ingress issues.

1. Introduction

Qatar currently has a construction boom taking place, largely, as a result of infrastructure requirements for the upcoming world cup in 2022, Qatar vision 2030 and large private sector investments. Keller has had a presence in Qatar since 2012 and has executed many grouting & excavation support solutions at the Doha metro underground project Doha Qatar. This paper highlights the scope of work and techniques Keller has utilized in executing these specialist geotechnical works i.e. ground anchors & permeation grouting.

2. General Overview

2.1. Doha Metro Qatar Integrated Rail Project

The Doha metro is a 85-km railway network and a part of Qatar Integrated Rail Project (QIRP). QIRP will include the east coast link, high-speed link, freight link and a light rail system. The railway will serve the suburbs of Doha and developments such as Lusail, Education City, and West Bay. There will be approximately 100 stations built for the entire Doha Metro Network and these will include two major stations built at Musheireb and Education City (see fig. 1).
2.2. Underground Conditions

The general underground conditions in Doha are shown in figure 2. There have been rare cases where open cut excavations in urban areas have had to utilize intensive dewatering schemes with wells, open trenches and local dewatering measures for this project.

The geological sequence of Doha are Quaternary and Tertiary. The residual soil has sand, gravel or clay as its predominant component and can be described as light brown, sandy, silty, gravelly, clayey, fine to coarse SAND/CLAY/GRAVEL with gravels that are fine to coarse sub-angular to sub rounded fragments of limestone. Residual soils are generally derived from physical and chemical breakdown of the underlying limestone bedrock. These materials are directly overlying the Simsima Member of the Eocene (upper Dammam Sub-Formation), Midra Shale, and Rus Formation.

Dissolution cavities can be occasionally encountered. Other small voids and fissures are also encountered in the limestone strata. When in proximity to the shore, these can be connected with the sea along greatly influence the local strata permeability.

Simsima Limestone can generally be described as unweathered to destructured (completely weathered). It is frequently fractured, crystalline dolomitic limestone with pockets of calcareous siltstone/silt. The upper layers of the formation are typically destructured which generally tend to be less competent in terms of cementation, strength, fractures and weathering with depth (RQD values vary from 0% to greater than 50%. The limestone has highly variable intact strength (UCS values vary from 5 MPa to 30 MPa and RMR vary from 11 to 55). It is also frequently intermixed, or contains pockets of very poorly to well cemented siltstone.

The Midra Shale Member consists two different facies including the Midra Shale/Siltstone and Midra Limestone. These layers can easily be identified in figure. These two facies are interbedded and sometimes
there may be up to three layers of siltstone with interbedded limestone. The thickness varies from 0 - 4m. The top and bottom of the unit are generally defined by the presence of the siltstone layers. This unit is highly variable in terms of intact strength (UCS values vary from 1 - 20MPa and RMR vary from 45 - 56).

The Rus Formation consists of very weak to weak, off white to dull, porous, fossiliferous, fine-grained limestone in the upper part of the formation, followed by the Lower part of it is Rus Formation Gypsum which is weak to strong, light to greyish brown, gypsum interbedded with limestone and claystone. This unit is highly variable in terms of intact strength (UCS values vary from 1 - 30MPa and RMR vary from 43 - 69).

3. Ground Anchors

Ground anchors are typically required as an essential part of the excavation’s lateral support system. Keller was assigned the task for installing these anchors at two major metro lines. The given specifications from the clients required various types of anchors to be installed. Due to a very extensive and generally complicated material-approval system, the supplier for the anchor system had to be defined in a very early stage of the project. Various suppliers from around the world have been involved with the international joint ventures of design and build contractors from the beginning stages of the project.

3.1. Types of Anchor Installed

In addition to the standard requirements, international building codes have necessitated the use of the “semi-permanent” anchor for all projects. This particular requirement defined a minimum design life time of 10 years for all parts of the anchors. The background of this requirement was to cover a possible time for an open excavation of more than 2 years duration.

Since the majority of the excavations were located in inner city areas, the property owners or authorities of the adjacent plots often required removable anchors for their properties. Therefore, most of the installed anchors needed to have the capability of being removed after reaching a certain point in the construction phase of the permanent structure.

At one metro line, anchors installed by Keller were mainly from DYWIDAG System International (DSI), though other brands such as MK4 and Dextra GFRP anchors have also been installed. For DSI and MK4, semi-permanent and removable or non-removable anchors have been installed (Fig. 3).
At a different metro line, all anchors installed have been from SAMWOO, and have a totally different load transfer mechanism for the removable and non-removable anchors. The non-removable anchor has been very similar to a permanent anchor from this manufacturer, except the head details. For permanent anchors, head details include a trumpet also for corrosion protection which was not present in anchors installed at this project hence called as semi-permanent anchor.

There are various differences detailed for several aspects of the anchors including the design of the individual parts of the anchor, details for corrosion protection, removal system for the anchors, details in the stressing procedures and the installation process itself. For removable anchors, three different systems have been utilized: Cutted wire, thermally weakened section and withdrawal by rotating. An experienced team of operators and engineers was required to ensure proper functioning of all anchors.

Figure 3 a-d shows the physical appearance of different kinds of anchoring strands used in this project.

![Some anchor types installed](image)

**Figure 3 a-d** Some anchor types installed

### 3.2. Red Line North

Red Line North is one of the two parts of the red line of the QIRP Project. Keller has been assigned as the specialist geotechnical contractor by the design & build joint venture for anchoring works. The work were completed at west bay station, Doha exhibition, Convention Center Station, Katara Station and at Exit Box near the Lagtia Golf Course. In addition, co-ordination and support in the detailed preparation for the execution of the excavation and a high flexibility in the anchor installation process has been provided for the project. For a typical cross section from the design reports please see Figure 4.

**Project Summary:**

- Started in November 2014 and finished in May 2015
- Approximately 1,250 anchors were installed with over 20,000m of drilling
- Supported wall systems: Diaphragm walls, bored pile walls, secant pile walls
- Anchor loads varied from 500kN to 1,500kN
- Length of anchors varied from 12m to 27m
The construction site is congested as shown Figure 5. Figure 6 shows the anchor drilling works for a station near the sea line. Its location lead to some serious issues due to the high water ingress.

The Gold line metro is running from the airport through the city centre towards south of Doha. Keller was assigned as the specialist geotechnical contractor by ALYSJ joint venture for design and installation of ground anchors for nine different metro stations. Figure 7 shows the typical cross section of anchors installed, whereas Figure 8 shows the execution of works for the third layer of anchors.

Project Summary:
- Anchoring works started in October 2014; finished by September 2015
- More than 1,500 anchors were installed with over 30,000m of drilling
- Supported walls: Diaphragm walls, bored pile walls, tangent pile walls, contiguous pile walls
- Anchor loads varied from 20kN to 2,000kN
- Anchor lengths varied from 12m to 45m
3.3. Anchor Design and Testing

The detailed anchor design has been carried out in accordance to EN1997. General anchor requirements (working and test loads, number of layers, spacing etc.) were determined from Excavation and Lateral Support (ELS) design of the station. The typical design value for the bond strength and length within the various layers of rock has been 300kN/m². This value has been verified during the various numbers of tests executed (see table 1).

As per ELS design, 100% of the anchors were tested for acceptance test as per EN1537. 2-3% of the anchors were tested for suitability and a few anchors on each station were designated as investigation test anchors. As per investigation load tests, the following skin friction values were observed in each type of strata. For design skin friction, a factor of safety of 2.0-2.5 was used.

Table 1 Typical case for Anchor test results and derived values

<table>
<thead>
<tr>
<th>Type of Strata</th>
<th>Drill diameter (mm)</th>
<th>Bond length (m)</th>
<th>Test Load (kN)</th>
<th>Max. Load achieved (kN)</th>
<th>Bond length shaft area (m²)</th>
<th>Calculated Skin Friction (kN/m²)</th>
<th>Design Skin Friction (kN/m²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Simsima Lime stone</td>
<td>150</td>
<td>3.0</td>
<td>1500</td>
<td>1500</td>
<td>1.413</td>
<td>1028</td>
<td>300</td>
</tr>
<tr>
<td>Highly weathered lime stone/Midra shale</td>
<td>178</td>
<td>5.0</td>
<td>1114</td>
<td>1114</td>
<td>1.413</td>
<td>778</td>
<td>300</td>
</tr>
<tr>
<td>Rus Formation</td>
<td>178</td>
<td>4.0</td>
<td>2000</td>
<td>2000</td>
<td>2.24</td>
<td>894</td>
<td>300</td>
</tr>
</tbody>
</table>

Loads for acceptance tests were defined at 125% of the design load. All anchors were tested first and then locked-off at the designated forces (Figure 9). As shown in figure 10 a carbon fibre reinforced polymer (CFRP) jack was used for pre-stressing with maximum loading capacity of 2,700kN. These pre-stress loads were also monitored by installation of load cells at few anchor locations on each station.

On some sites waling beams were also installed to distribute loads uniformly over a group of piles. For different anchor types, different pre-stressing protocols were developed.
3.4. Installation of Anchors

The standard execution process as per EN1537 was followed for ground anchors. Anchors were drilled with a diameter of approximately 150mm. Due to the highly fractured rock zones, the drilling need to be completely cased. Finally the anchors were then installed into the boreholes. Boreholes were bottom-up grouted before inserting the anchors. Pre-assembled anchors were fitted with grout injection tubes in a certain arrangement, lowered into boreholes before the casings were retrieved. Primary and secondary grouting were then carried out. This technique has been the key to the highly successful installation of anchors.

In all cases dewatering measures (deep wells around and inside the excavation) were in place during the execution of the anchors. Due to the nature of the highly fractured ground, some cases of water ingress of up to 300 litres/second was recorded. Even though such numbers are exceptional, in many locations “normal” water inflow of 10-30 l/sec from an individual bore hole had to be managed. To stop such water ingress, pre-assembled anchors were fitted with multiple packers and injection tubes and that would seal the borehole. Pressure grouting was done in later stages to ensure proper grout in the bonded length. A detailed pressure and volume criteria was devised for quality control. During the initial phase of anchor installation, a detailed plan of the active dewatering system was developed to minimize the possibility of a negative effect of the water flow on the final capacity of the anchors. Figures 11, 12 and 13 show some situations to be dealt with on site.
4. Grouting Work

Due to the rocky strata of the Qatar Peninsula, permeation grouting was considered by designers as the primary solution for water control at Doha metro project. Keller executed permeation grouting work at both the Gold Line and Red Line south projects. The purpose of these measures have been to reduce the water inflow. Therefore, the entire water management system typically consisted of active dewatering using deep wells both outside and inside the excavations. This system decreased permeability by executing the permeation grouting and French drains for the collection of the water, inside the excavation (Figure 14).

Figure 11 “Normal” water ingress during anchor installation

Figure 12 & 13 Some particular situation to be dealt with to ensure the capacity of the anchor
Due to the highly variable nature of fissures/cavities in the sub-surface throughout the region, performance criteria was to ensure effectiveness of the grouting curtain. The grouting technique employed was bottom up grouting. Borehole plans were divided into primary boreholes, secondary boreholes & tertiary boreholes. Each borehole in depth was sub-divided into different stages. Each stage was sub-divided into steps. Grout volumes and pressures were monitored and recorded to an entire overview for the individual locations. Such grout consumptions were depicted in longitudinal sections. Zones of high consumption were indicative of high fracture zones. The continuous evaluation of grouting data assisted in treating the ground with the right grout mixes and quantities. The design concept of grouting curtain is shown in figure 14.

The Grout curtains were of various water cement ratio which varied from 0.75 to 1.75. Admixtures and accelerators like bentonite and sodium silicate were added to improve the grout properties (e.g. setting time, bleed etc.).

One critical point for the effectiveness of the grouting works has been the fact that during the execution of the grouting, the active dewatering process was already functioning. This affected the general time schedule due to the adopted grouting technique and was taken into consideration by the clients and their designer.

In addition, in several locations additional grouting works were required in order to stop local water ingress or to allow maintenance work for the cutters of the tunnel boring machines. Depending on the requirements, pure cement grouting or grout with additives had been used.

**Figure 14** Design model with grout curtain for reduction of water pressure (from ELS design report)

### 4.1. Standard grouting works at Gold Line

For metro stations up to 40m x 800m size (Figure 15) permeation grouting using Tubes a Manchettes (TAM) have been executed from the surface. The TAM with bottom-up grouting stages had been preferred to avoid extensive drillings works which would be required for the top-down approach.
The grouting itself has been executed using standard technologies like single and double packers, colloidal mixers and piston pumps installed in a grouting container. Typically, three different mixes have been used. For the grouting process, clear pressure and volume criteria have been defined depending on the overburden, water levels and in-situ rock conditions. Drilling logs & grout consumption in primary boreholes reveals the efficacy of selected pressure and volume criteria.

Three metro stations were grouted with a grout curtain design and one metro station grouting works was carried out to mitigate excessive water ingress into the excavation box.

Project Summary:
- Grouting works commenced in August 2014 and were finished in April 2015
- Drilling and TAM installation of more than 30,000m for maximum depth 30m
- More than 4,000m$^3$ of grout volume
- Standard spacing between primary boreholes was 10m; secondary boreholes were completed after evaluation of the drilling and grouting data

Figure 15 Open cut with grout curtain

Due to high temperatures of up to 55°C during in the summer, the grouting works were mainly executed during the night shift (Figure 16). Otherwise, permeation of grout into fissures/fractures would have been restricted and ultimately would have reduced grouting efficiency. Figure 17 shows how some fractures above the excavation level were filled with grout.

Figure 16 Grouting works at night shift

Figure 17 Fissures filled with grout

4.2. Remediation Works – Flooded Station

Keller was engaged in this project for mitigation of a flooded excavation pit. The original design was open cut with active de-watering using deep wells. Huge discharges of water could not be dealt with as the excavation proceeded. The dewatering amount went up to 2,000m$^3$/hr, from a design value of 700m$^3$/hr,
even at only 10m excavation level. Keller used TAMs to create a grouting curtain against the water inflow into the excavation. The grouting work has been concentrated in the Simsima limestone which had a high permeability. This resulted in a reduction of approximately 30% of water inflow therefore enabling the client to excavate down to the design depth (Figure 18).

Project Summary:

- Grouting works commenced in January 2015 and were finished in February 2015
- Drilling and TAM-installation of more than 7,500m
- More than 400m³ of grout volume was pumped
- Standard spacing between primary boreholes was 3m
- Secondary boreholes were completed after the evaluation of the drilling and grouting data

![Figure 18 Excavation works after grouting and image of untreated excavation box](image)

5. Quality Control and HSE

The grouting process for the project required a very strict quality control and assurance plans. This quality assurance involved material quality e.g. viscosity of grout, material type, free water determination, strength of grout, etc. For drilling, a detailed checklist had to be followed which included drilling logs, grout consumption, logging and geometrical parameters. Regular calibrations, certification of the plant, equipment and personnel ensured the quality of the process. A detailed risk assessment including mitigation measures were also in place.

Keller executed all projects as per Keller’s and the client's strict HSE policies. All hazards related to anchoring and grouting activities were identified and mitigated. Risk assessments were prepared for each activity ranging from drilling to the handling and lifting of rods, casings and materials, grouting works, pressure hoses, equipment handling, material handling, pre-stressing, etc. All hazard ratings were reduced to acceptable values. Training for workers, operators, supervisors and on-site management staff were conducted regularly. Environmental and personnel health and safety concerns were fully monitored and corrected when necessary.

Furthermore, the gathered data had to be evaluated directly on site to ensure a high quality grouting process. Therefore, simple tools have been developed to enable the geotechnical lead engineers to make quick, sound decisions about potential additional grouting works in certain locations. Some examples of the
evaluated data and submitted sheets are shown in figures 19 and 20. The various red colours shown in the below figure indicate the grout consumption per stage and ground. A combined evaluation of the collected data from drilling and grouting has been the base to decide if additional grouting works have to be done.

<table>
<thead>
<tr>
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<th>No.</th>
<th>Average [litres]</th>
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<td>130</td>
</tr>
<tr>
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<td>256 No.</td>
<td>993</td>
</tr>
<tr>
<td>10 to 15 m</td>
<td>347,653</td>
<td>258 No.</td>
<td>1,347</td>
</tr>
<tr>
<td>15 to 20 m</td>
<td>196,797</td>
<td>256 No.</td>
<td>1,538</td>
</tr>
<tr>
<td>20 to 25 m</td>
<td>404,585</td>
<td>258 No.</td>
<td>1,558</td>
</tr>
<tr>
<td>25 to 30 m</td>
<td>67,360</td>
<td>76 No.</td>
<td>888</td>
</tr>
<tr>
<td>Sum</td>
<td>1,903,212</td>
<td>1,358 No.</td>
<td></td>
</tr>
</tbody>
</table>

Figure 19: Data preview in longitudinal section and in plan

6. Conclusions

A sound combination of experience, innovative thinking, available resources, optimized design and high QHSE standards during execution enabled us in delivering successful geotechnical solutions to these discussed projects. Multiple challenges were faced, but solved through proven ground engineering techniques which enabled project completion on time.

7. Acknowledgement

Keller would like thank all participants on the presented projects for their successful cooperation and team spirit.
In-situ dynamic testing of rock support at LKAB Kiirunavaara mine

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Abstract

A series of large scale dynamic tests were conducted at the LKAB Kiirunavaara mine using explosives to generate the dynamic load on the support system. This was done with the aim of developing a testing methodology for in-situ testing of ground support. Furthermore, the response of the installed rock support system to strong dynamic loading was evaluated. The results of the Tests 1, 2, 3, 4 and 5 indicated that the relation between the burden and the used amount of explosive had a vital role in either reducing or involving the effect of the detonation gases in the test results. Higher peak particle velocities were measured compared to those of similar large scale tests carried out in other countries. However, the level of induced damage in Tests 1 and 2 was limited to a fractured zone behind the support system while in Tests 4 and 5 the burden was unexpectedly destroyed. Based on the test results and preliminary numerical analysis, a modified test (Test 6) was designed at the same mine. The aim was to avoid the unexpected damage of burden as was observed in earlier tests, and to modify the dynamic loading leading to increase the depth of fractured zone and if possible pushing the support system beyond its limit. Results indicated that a larger fractured zone compared to earlier tests was developed behind the support system while the installed support system was still functional. Evidence from the damage to the tested cross-cuts in Test 6 indicated a reduction of radial cracks that provide access for the gas expansion. The results indicated that the installed support system, designed for dynamic conditions, performed well under the loading conditions which can cause ejection.

1 Introduction

In the design of support systems under seismic loading conditions, neither the demand nor the capacity of a support system can be satisfactorily defined leading to a case of design indeterminacy (Stacey 2012). Therefore, in order to quantify the performance of the rock support systems suitable for dynamic loading conditions, four main types of dynamic tests are considered including simulated large scale experiments by means of blasting, drop test facilities that apply an impact load on the reinforcement, laboratory tests applying dynamic loads on core samples, and passive monitoring and back analysis of case studies (Hadjigeorgiou and Potvin 2007). There are advantageous and disadvantageous with any of these approaches, but the provided data by different methods are complementary leading to the development of the data base for dynamic rock support performance (Hadjigeorgiou and Potvin 2007).

Within the framework of a research program focused on deep mining problems at Luleå University of Technology, in-situ dynamic testing of rock support using blasting as the seismic source was conducted in the Kiirunavaara underground mine, owned and operated by Luossavaara Kiirunavaara Aktiebolag (LKAB). The main objective of the tests conducted was to develop a large-scale in-situ testing method for evaluating rock support performance. This was done by exposing the rock support system to seismic waves generated by means of blasting. For this purpose, a series of large scale tests (Tests 1 to 7) were carried out in pillars/cross-cuts in the northernmost part of the Kiirunavaara mine. Results from Test 1, 2, 4 and 5 are
presented in detail in Shirzadegan et al. (2016b), and Test 6 in Shirzadegan (2016a). This paper presents a summary of the all conducted 7 tests.

It should be noted that there are certain differences between loading conditions from a seismic event and a simulated rockburst by means of blasting. A real seismic event of shear type first initiates a compressive wave followed by a larger amplitude shear wave which carries more energy than the longitudinal wave. The event-induced waves comprise all frequencies but the high frequency components are rapidly attenuated with travelling distance while the low frequency components attenuate with a much slower rate with distance. This means that ejection is most likely to occur close to the seismic source while significant dynamic stress contributions (tensile and compressive) leading to damage caused by superimposed static and dynamic stresses can be expected at all distances from the seismic source. This is also true for the shaking damage mechanism. Blasts produce waves which are dominated by P-waves with higher dominant frequencies than those induced by real seismic events. It was therefore decided to focus on simulating the ejection damage mechanism using blasting as the seismic source.

Large scale seismic event simulations have been performed in different parts of the world in order to assess the performance of ground support systems since 1969 (Andrieux et al. 2005, Ansell 2004, Archibald et al. 2003, Espley et al. 2002, Hagan et al. 2001, Heal 2010, Heal and Potvin 2007, Heal et al. 2005, Ortlepp 1969, Tannant et al. 1994, Tannant et al. 1995, Ortlepp 1992). Different blast layouts (e.g. blasthole angle and burden) were used by the different researchers based on the objective of their tests. Different levels of success in obtaining the desired amount of damage to the rock support /rock mass were observed.

Stacey (2012) reviewed in-situ testing methods of rock support. He divided the design of the tests into the categories direct and indirect testing methods. Direct testing of rock support is considered as a severe test of rock support where the damage to the support system is directly due to the blast. In this method, the gas pressure has a strong influence which makes such a test very different from a rockburst. The test carried out by Ortlepp (1992b) was an example that falls within this testing category. Ortlepp (1992b) compared the damage from a real rockburst that occurred close to the site where the test was conducted. He concluded that the damage produced in a direct test is indistinguishable from the rockburst damage. Stacey (2012) concluded that, the results provided by this method should be compared to documented real rockburst damage, perhaps using a damage classification approach.

In indirect blasting tests the issue of gas pressure is almost completely solved by increasing the burden (Stacey, 2012). This type of test has been considered as the closest simulation of rockbursts. However, there are still questions that need to be resolved such as the difference in wave interaction, wave frequency, source mechanism, source location, and source magnitude. Further issues indicated by Stacey (2012) are that the tests are not repeatable at one site, they are too costly, and too inconvenient in an underground mine environment to be practical for comparison of the performances of different support systems.

A summary of some earlier tests with direct, indirect or a combination of them is presented in this section. In the tests carried by Andrieux (2005) and Tannant (1994) angled blastholes were drilled into the wall of the test drift (Figure 1). The main objective was to separate the effect of waves from the detonation gases. The angled geometry of the blastholes provided (according to Andrieux et. al (2005)) an easier path for the detonation gases to vent backwards. Only the part of the wall close to the collar was destroyed and small signs of damage were observed at the charged segment of the blasthole in all tests except in the second trial where the whole test wall was destroyed. Tannant (1994) described that the blasthole relative to the test wall in their tests was designed to generate a variation in the blast loading intensity along the tested wall.
In the tests conducted in South Africa (Hagan et al. 2001) a large burden of 5 m was considered to minimize the effects of expanding gases. Figure 2 shows a picture of the damage to the sidewall and the volume of the blocks ejected after the simulated rock burst. Reddy and Spottiswoode (2001) showed that the shape of the ejected blocks was determined by the pre-existing bedding separations and stress fractures. Post blast observations indicated that no direct gas expansion had affected the ejection of the rocks (Hagan et al. 2001).

In the tests conducted by Heal and Potvin (2007), three blasts were conducted at one site, with a burden of 5 m, 4 m, and 3 m in the first, second and third blast, respectively. Each blasthole was separately charged and detonated to allow successively larger dynamic loading upon the test wall. In these tests, successive blasts were designed to reproduce an actual seismic event as closely as possible.
2 Tests 1 to 5

2.1 Design

Considering the mentioned difficulties and uncertainties associated with direct and indirect blasting methods including the penetration of explosive gases into the burden, the issues with the differences in loading conditions and source mechanism, and the cost of the tests which are all believed to be common in both approaches, it was decided to design the tests in the Kiirunavaara mine to possibly overcome some of the abovementioned problems. The aim was to design the tests so the rock support system could be tested with one blast and to ensure that the loading of the rock support was caused by seismic waves and thus the effect of the blasting gas had to be minimized. For this purpose, one blasthole was drilled in the pillar parallel to the test wall and four holes were drilled into the burden to measure the gas pressure in the burden.

A high VOD explosive type, NSP711 with VOD around 7900 m/s, was selected for the tests. The reason for selecting this type of explosive was the lower amount of gas production compared to commercial explosives. The only exception was Test 3 in which bulk emulsion was used.

In Test 1, the blast was designed to generate a magnitude +3 event (Richter scale) located 15 m from an opening. This would result in PPVs in the range of 1.5 m/s to 3.5 m/s according to the PPV – magnitude – distance relationship by Kaiser et al. (1996). The design magnitude was similar to that of the largest seismic event that had occurred in the Kiirunavaara mine until 2010 (Malmgren 2010). The burden and blasthole diameter in Test 1 were determined based on the experience from earlier studies in the Kiirunavaara mine by Olsson et al. (2009). This resulted in a theoretical burden of 3.5 m and a blasthole diameter of 115 mm. Two different charge concentrations, \( d_c^1 = 76 \text{ mm} \) and \( d_c^2 = 45 \text{ mm} \), denoted as “high charge segment” and “low charge segment”, respectively, were used. The low charge segment was the first 5 m of the blasthole measured from the toe. The high charge segment was the subsequent 5 m of the blasthole. This was done in order to investigate the support performance under two different levels of energy during one test and reduce the number of trials in the search for the optimum blast design. The total length of the blasthole was 15 m. The last 5 m of the blasthole, from the high charge segment to the collar, was left empty.

The blast in Tests 2 to 5 were designed based on the observed results in the earlier tests. The burden in all these tests was kept constant while the amount of explosive was changed based on the experienced level of damage in previous test. Figure 4 shows a schematic plan view of the test sites and the blast design in Tests 1 to 5.

2.2 Rock support and instrumentation

The rock support system used in all tests comprised 100 mm steel fibre reinforced shotcrete (40 kg/m\(^3\) steel fibre), 75 mm × 75 mm welded mesh with 5.5 mm diameter, and Swellex Mn24 rockbolts with a length of 3 m and 1 m spacing.

The instrumentation was designed to provide data for different objectives. As an example of the instrumentation, the monitoring layout in Test 1 is shown in Figure 5. The exact number of monitoring instruments in each test is summarized in Table 1. The reason for not using monitoring instruments in Test 3, 4 and only a few instruments in Test 5 was to speed up the process of finding the optimal burden and charge concentration.

The instrumentation was designed to provide data to assess the performance of the rock mass and the rock support system. For this purpose, uniaxial shock accelerometers, type PCB 350 B03, were used to measure the acceleration and to calculate the velocity and the displacement. Observation boreholes, inspected by Robicam 37 before and after the blast, were used to measure the depth of the damage created behind the surface of the test walls.

High-speed cameras (Casio EX-F1) were used in each test to record the response of the tested walls and to estimate the ejection velocity where possible.
Laser scanning (Leica HDS 6000) was conducted before and after each blast to measure the residual displacement of the test walls due to dynamic loading. The scanning also provided information on the deflection of the reinforced shotcrete. The measured deflections were then used to estimate the energy absorption by the reinforced shotcrete.

In some of the tests, a laser sensor type Sick OD Value Sensor was placed in the middle of the cross-cut to record the displacement versus time of the test walls during the blast. The aim was to study the dynamic response of the tested walls under different designs.

Figure 4  Blast design in Tests 1, 2, 3, 4 and 5 (Shirzadegan et al. 2016b)

Figure 5  Monitoring instruments lay out in Test 1 (Shirzadegan et al. 2016b)
Table 1  Summary of the number of monitoring instruments in Tests 1, 2, 5, 6, and 7

<table>
<thead>
<tr>
<th>Test</th>
<th>No. of accelerometers</th>
<th>No. of observation boreholes</th>
<th>No. of laser sensor</th>
<th>Laser scanning</th>
<th>Gas pressure sensor</th>
</tr>
</thead>
<tbody>
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<td>4</td>
<td>-</td>
<td>Yes</td>
<td>Yes</td>
</tr>
<tr>
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<td>24</td>
<td>4</td>
<td>2</td>
<td>Yes</td>
<td>Yes</td>
</tr>
<tr>
<td>5</td>
<td>3</td>
<td>-</td>
<td>1</td>
<td>Before blast</td>
<td>No</td>
</tr>
<tr>
<td>6 crosscut 100</td>
<td>16</td>
<td>11</td>
<td>1</td>
<td>Yes</td>
<td>No</td>
</tr>
<tr>
<td>6 crosscut 103</td>
<td>5</td>
<td>9</td>
<td>1</td>
<td>Before blast</td>
<td>No</td>
</tr>
<tr>
<td>7 crosscut 100</td>
<td>16</td>
<td>11</td>
<td>-</td>
<td>No</td>
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</tr>
</tbody>
</table>

2.3 Results

In Test 1, the post-blast damage mapping showed a few discrete cracks in the shotcrete but no damage to the mesh or the rockbolts was observed. The range of the measured PPV recorded by the accelerometers was between 1 and 6.7 m/s. The depth of the created fractured zone behind the shotcrete measured by the borehole camera was 0.3 to 0.4 m. The kinetic energy transmitted to the rock support was estimated to be in the range of 6.0 - 20 kJ/m$^2$.

The level of the input energy for Test 2 was increased by 50% compared to Test 1. Test 2 created larger number of, longer and wider cracks than in Test 1 and the cracks were distributed all over the tested wall. However, no damage of the rockbolts was observed despite the high PPV (up to 7.5 m/s measured by accelerometers). The depth of the created fractured zone was 0.5 m and the transmitted kinetic energy was in the range of 2.0 - 30 kJ/m$^2$.

Test 3 was carried out at the same location as Test 2 using the same blasthole. Therefore the burden was already fractured due to the blast in Test 2. Unlike the rest of the tests, the blasthole in Test 3 was filled with bulk emulsion. A complete destruction of the burden occurred in this test. The broken rock pieces were ejected and thrown to the other side of the cross-cut. Mapping of the rock pile showed that some of the ejected rockbolts did not show severe damage probably indicating that the distance from the tested wall surface to the release surface of the burden was greater than the length of the bolts (Figure 6a). Some of the rockbolts were severely damaged, mainly either deformed at 2 or 3 cross-sections along their 3.0 m length or broken into pieces of lengths from 0.2 to 3.0 m. The broken rockbolts showed signs of shear failure at the failure surface. The face plates were in a few cases detached, in some cases severely deformed indicating that they had been highly loaded, and in some cases remained undamaged. However it was difficult to judge with high certainty that the observed damage to the rockbolts was due to the blast load, gas pressure or a result of becoming broken under the rock pile after ejection.

In Test 4 the input energy was increased by 10% compared to Test 2. The burden was the same as in the earlier tests. The increase of charge concentration resulted in a complete destruction of the burden. Figure 6b shows the damage to the rockbolts in Test 4. The charge concentration for Test 5 was in between that of Tests 2 and 4 (dc$_1$ = 94 mm and dc$_2$ = 83 mm in a 16 m long blasthole). Complete destruction of the burden occurred in this test. The damage observed to the rockbolts in Test 4 and 5 was similar to that observed in Test 3.

A relatively high pressure (about 800 kPa) was recorded by one of the gas pressure sensors (approximately 1 m from the blasthole) in Test 1, 34 ms after the initiation of the detonation and a low pressure was recorded by the rest of the sensors in both Tests 1 and 2.
3 Analysis of the tests and improved blast design

3.1 Tests 1 to 5
Tests 2 and 5 were numerically analysed using a combination of LS-DYNA and UDEC (Zhang et al. (2013). The LS-DYNA was used to simulate the detonation stage of the blast. The velocity-time history calculated by LS-DYNA was applied as an internal boundary condition in UDEC. The simulation of Tests 2 and 5 resulted in thin yielded areas parallel to the tested walls near the free face and a large number of thin radial yielded areas from the crushed zone boundary created around the blasthole, forming a large cone-shaped volume. The conically shaped rock mass moved towards the cross-cut, with extensive yielding occurring along and within this volume. The results from the numerical analysis of Test 5 were quite representative of the conditions after Test 5 in which the test wall and part of the pillar were completely destroyed. The conclusion of Tests 1, 2, 3, 4 and 5 was that the loading condition needs to be improved to avoid the creation of radial cracks and the conical failure volume and to promote the generation of dominantly wall-parallel fractures mimicking the damage caused by a planar seismic wave.

3.2 Improved blast design (Test 6)
Numerical simulations with larger burdens, i.e., 5 m and 8.5 m (larger than in Tests 1, 2, 4 and 5 and the largest possible at the test site), were therefore conducted by Zhang et al. (2013). The number of blastholes was also increased to two in this analysis with the aim to generate a wave with more planar front, and increase the strength of the designed blast to develop a larger failure zone behind the test walls compared to earlier tests. The results from the numerical analyses showed that fractures parallel or sub-parallel to the wall surface were created without forming any cone-shaped volume for both 5 m and 8.5 m burden. Based on the results from the numerical analysis it was decided to use a burden of 8.5 m for Test 6. Therefore, two blastholes with zero delay and a designed burden of around of 8.5 m was proposed in order to generate a wave front which is sub-planar wave to the surface of the tested wall and to reduce the negative effect of radial fracturing, and increasing the strength of the blast. Figure 7 shows the design specifications in Test 6. The blastholes were drilled in the middle of the pillar between cross-cuts 100 and 103. The tested wall in cross cut 100 was supported by the support system mentioned in section 3 while the tested wall in cross-cut 103 was supported by only plain shotcrete.
3.3 Results

The lower blasthole did not detonate during Test 6. No major damage to the rockbolts and mesh was observed in cross-cut 100, after the blast. The shotcrete, showed a few very fine cracks. Clearly evident damage to the surface support, i.e., debonding of the shotcrete from the rock over an area of roughly 1 m x 1 m was observed at the end of the test panel (farthest from the footwall drift, see Figure 8a). However, the damaged material was kept in position by the mesh. Farther into the cross-cut, the areas not covered with the dynamic rock support, showed ejection of wall material with a depth of up to 30 cm. The range of the measured PPV in this cross-cut was between 1.3 and 4 m/s. The depth of the created fractured zone behind the shotcrete measured by borehole camera was 0.3 to 0.6 m. The maximum transmitted kinetic energy in this cross-cut was 10 kJ/m².

Cross-cut 103 failed to be functional and blocks of rock were ejected from the tested wall by the blast (Figure 8b). The large pieces of rock and shotcrete lying in the middle of the cross-cut were ejected from a location of 1.5 to 2 m above the floor and travelled a horizontal distance of about 2 m. The failed thickness per square meter of the test wall (range 0.1 – 0.8 m) was measured by direct observation after removal of the rock piles from the cross-cut. Based on analysis of the video recorded by a high speed camera, an ejection velocity of 3.6 - 8.1 m/s was estimated in this cross-cut. The maximum transmitted kinetic energy in this cross-cut was 29 kJ/m².

![Figure 7](image-url)  
**Figure 7**  Schematic view of the blast design in Test 6 (Shirzadegan et al. 2016a)

![Figure 8](image-url)  
**Figure 8**  Damage to cross-cut 100 after (Shirzadegan et al. 2016a)
3.4 Test 7

Since the lower blasthole did not detonate in Test 6, it was decided to recharge the upper blasthole and blast both holes a second time (Test 7). A similar number of accelerometers were installed in cross-cut 100. Post blast observations indicated that the number of cracks on the surface of the shotcrete increased. No significant damage to the rockbolts and weld mesh was observed. Lower range of particle velocity compared to Test 6 was recorded by the accelerometers (1-3 m/s). This could be due to the fact that the burden was fractured by the blast in Test 6.

4 Rock support performance

4.1 Swellex Mn24

The deep installed accelerometers, 1.5 m from the surface of the shotcrete, at each charge section in Test 2 were used to estimate the performance of the rockbolts. The Swellex rockbolts closest to these accelerometers were selected for analysis of energy absorption versus displacement. A maximum elongation of 80 mm was estimated for the Swellex Mn24 in Test 2. The results from a series of dynamic laboratory tests on Swellex Mn24 by Voyzelle et al. (2014) was used to estimate the amount of absorbed energy by the Swellex rockbolts in Test 2. The estimated elongation was used in combination with the energy absorption curves by Voyzelle et al. (2014). This resulted in an energy absorption of around 17 kJ for the Swellex MN24 for an elongation of 80 mm.

4.2 Fibre reinforced shotcrete

At the points of the tested wall where rockbolts were installed, the relative displacement between the rockbolt and its surrounding surface support was calculated to estimate the maximum residual deflection of the surface support after the dynamic loading. The displacements of the rockbolt and surface support were obtained from the laser scanning of the test wall before and after the blast. The measured deflection of the shotcrete for each square meter of the tested wall was used to estimate the energy absorbed by the reinforced shotcrete and the weld mesh in Tests 1, 2 and 6 (cross-cut 100). The corresponding absorbed energy per square meter of the tested walls was then estimated by using the deflection-energy absorption curves by Thyni (2014) from a series of laboratory tests of reinforced shotcrete and weld mesh at the LKAB Kirunavaara mine. Figure 9 shows the energy absorbed by the reinforced shotcrete and weld mesh per square meter of the test wall in Test 2. The range of energy absorption in the mentioned tests is:

- Test 1: 0.1 – 0.8 kJ/m²
- Test 2: 0.2 – 4.0 kJ/m²
- Test 6: (cross-cut 100): 0.3 – 0.8 kJ/m²

![Figure 9](image_url)

Figure 9 Energy absorbed by reinforced shotcrete and weld mesh in Test 2 (Shirzadegan et al. 2016b)
4.3 Support system

The maximum calculated kinetic energy of the ejected rock (29 kJ/m²) in cross-cut 103, according to the classification by Kaiser et al. (1996), falls within the range of very high level damage intensity. As the blasthole in Test 6 was drilled almost in the middle of the pillar it could be expected that the blast will result in similar damage intensity in cross-cuts 100 and 103 if the geology is the same and the same support is used. Cross-cut 103 was supported with plain shotcrete while cross-cut 100 was supported with a dynamic rock support. Therefore, Test 6 provided an opportunity to compare the damage intensity and the recorded PPV (by accelerometers) for an unsupported cross-cut and a cross-cut supported by Swellex Mn24 + fibre reinforced shotcrete + weld mesh. The generated average PPV 0.2 m behind the surface of the tested wall in cross-cut 100 where the dynamic rock support system was installed was up to 1.5 times lower than that in the unsupported cross-cut 103. The maximum transmitted kinetic energy to the fractured zone in the test wall in this supported cross-cut (100) was estimated to 10 kJ/m². Except minor cracks visible on the surface of the shotcrete, no sign of damage jeopardizing the supporting capacity was observed. Based on the kinetic energy measurements and site investigations after the blast in the two cross-cuts, it can be concluded that, the combination of Swellex Mn24, reinforced shotcrete and weld mesh can mitigate the consequences of a seismic event where the very high level of damage intensity by rock ejection is expected.

5 Discussion

Tests 3, 4 and 5 resulted in severe damage to the support system. The burden was completely destroyed, and the rockbolts were ejected and exposed. The numerical analysis revealed that using high amount of explosives and a burden of 2.5 m to 3.5 m has resulted in tangential tensile stresses exceeding the tensile strength of the rock and a reduction of the radial stresses close to the wall of the cross-cut.

In Test 6, the burden was increased in order to generate a wave front which was sub-planar to the surface of the tested wall. The improvement of the design was confirmed by the observed damage to the tested wall in cross-cut 103, i.e., ejection of rock slabs with a thickness of up to 80 cm. The whole 8.5 m burden was not destroyed despite the fact that this cross-cut was not supported with the dynamic rock support (only plain shotcrete was sprayed).

In Tests 1, 2, 6 and 7, despite the low level of damage to the support, more quantified data was produced compared to that in Tests 3, 4 and 5. This included, the PPV, the depth of damage, and the estimated kinetic energy transmitted to the support system. Furthermore, the tests provided calibration data for use in numerical models. The tests also provided the possibility to interpret the performance of the rock support in Tests 1, 2 and 6. This was done by linking the data obtained from these tests to the data provided from laboratory dynamic tests conducted on the rockbolts and surface supports.

In Test 6 the damage to the tested wall in cross-cut 103 was similar to that which occurs in a rockburst with high damage intensity (according to classification by Kaiser et al. 1996). As the blasthole was drilled in the middle of the pillar in this cross-cut, it could be expected that the test wall in cross-cut 100 was similarly loaded. Therefore it can be concluded that, the support system in cross-cut 100 has withstood the dynamic loading conditions. It can be concluded that, the installed support system can limit the damage and enhance the safety of the excavation where high damage intensity from a rockburst is expected. It is believed that this method will provide better opportunity to compare different support alternatives that could be used at a mine site under seismic loading conditions.

One of the goals for the large scale testing of rock support was to assess the energy demand on the tested support system for the design purposes. The tests conducted in the Kiirunavaara mine provided the opportunity to study the response of the test wall under dynamic loading and evaluate the performance of the installed rock support. It was decided to not continue with the tests after Test 7. Therefore no design data was provided from this testing series.
6 Conclusion

This paper presented a summary of the results from the development of a large-scale dynamic testing method of rock support systems by using blasting to generate the dynamic load. The major conclusions of this work are:

- The influence of gas expansion on the test results can be reduced by using high impact, low gas explosives, avoiding stemming the blasthole, and designing a reasonable burden.
- Evidence of damage showed that the new blast design (Test 6) was successful in generating sub-planar waves with tangential tensile stress levels lower than the tensile strength of the rock mass. Thus it can be concluded that no radial cracks with an extension from the blasthole to the tested wall surface was created.
- The installed support system (in Test 6) showed that it has the capacity to perform well in the mining areas where very high ejection conditions are expected.
- The large amount of data recorded during these trials will be useful for the calibration of more advanced numerical models. The numerical analyses can then be used for sensitivity analyses simulating different blast designs.

Acknowledgement

This work was financially supported by LKAB, Boliden, Centre of Advanced Mining & Metallurgy at LTU (CAMM) and the I²Mine project (an EU 7th framework project) which are gratefully acknowledged. The LKAB Mining Company in Kiruna is gratefully acknowledged for providing the opportunity to conduct the tests at this mine.

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Integrating microseismic and 3D stress monitoring with numerical modeling to improve ground hazard assessment

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S. Mozaffari, A. Nyström, P. Fjellström, BOLIDEN, Sweden

Abstract

INERIS and BOLIDEN are developing at Garpenberg mine (Sweden) new methodologies to monitor and to assess both quasi-static stress changes and ruptures in a seismic-prone area subject to deep mass-mining production. To achieve monitoring, a local mine seismic network has been deployed in the beginning of 2015 in the Lappberget area between 1100 and 1250 meter depth, in addition to 3D stress monitoring cells. Such network has been designed to fit with the sublevel stoping method and with the paste fill production/distribution system used by Boliden. Geophysical and geotechnical data are acquired continuously and near-to real time transferred to INERIS data centre through e.cenaris e.infrastructure for automated data processing and database management, along with mining datasets. In addition to continuous monitoring, a fine-grid 3D numerical model of the mine has been defined, in which the complex 3D shapes of the orebody and a major weakness unit are taken into account. It is submitted to successive step-by-step (exploited + backfilled stopes) simulations to assess the stress variations following a correct stress path, the plastic state, the safety factor, the strain and displacement fields, both elastic and plastic (shear and volumetric strain) energies in the mine in response to its development as stresses are monitored for quantitative comparison. Until November 2015, up to 20.000 m$^3$ of ore were extracted within the monitored area, highlighting numerous stress shifts ranging from 0.1 MPa to 15 MPa with the stress cells. Methodology and data presented here are the preliminary results of the first year of monitoring. The study focuses on 3 main stress shifts recorded by the stress cells at level 1157, which are associated with the recorded seismicity, and then compared to the numerical model. Indeed significant immediate and differed stress shifts associated with induced microseismic events are recorded over a several-week period following the first blasts. In addition, modeling reveals that the presence of weak horizon (talc) influences the dynamic response of the mine by inducing creep and plasticity phenomena, which would explain the results obtained by geotechnical measurements and microseismicity.

Keywords: microseismic monitoring, stress shift, modeling, risk assessment

1 Introduction

Nowadays, the underground mining industry has developed high-technology mass mining methods to optimize productivity at deep levels. Massive extraction induces high-level stress redistribution, which may generate seismic events around mining works (Gibowicz and Kijko 1994). Numerous authors have already highlighted the necessity of steadily enhanced scientific practices and technologies to face mining-induced seismicity and rockburst hazards, in order to guarantee the safety of both miners and the mining environment (Ortlepp 1997, Durrheim et al. 2006, Durrheim 2010, Hudyma and Potvin 2010). In this context, Hudyma and Potvin (2010) have proposed an engineering approach based on the application of microseismic monitoring to identify the active seismic sources, to assess the related risks, and to implement suitable mitigating measures. Most of these monitoring activities have been successfully applied in deep mines in Canada, South Africa, and Australia (Hudyma and Potvin 2010) where production depths easily reach 2000 m and much deeper in South Africa. Similar applications are also being employed in European mines subject to deep-production stress shift (Lizurek et al. 2014, Szczersbowski et al. 2015 in Poland, Janiszewski 2014 in Finland).
Geotechnical monitoring is currently used in numerous mines through the use of extensometers, crackmeters, etc. Maleki 1990 presents an example of a stability evaluation of a coal mine based on modeling coupled with borehole pressure cells and deformation measurements. Studies realized by Potvin and Wesseloo (2013) present a well-documented example of instrumented mine in Tasmania with implementation of seismic network and CSIRO HI cells measurements, showing a strong correlation between the mining blasts, large seismic events and the stress changes measured by close instruments. Nevertheless, geotechnical sensors provide accurate information on aseismic deformations locally and depending on the mining method they may be costly to deploy.

Eventually, monitoring the mining process is essential to fulfill a geohazard assessment routine based on geotechnical and seismic methods. Indeed, huge-stress transfers and rock failures are generated by the mining process itself around the freshly created excavation. High-quality technological mine data is nowadays becoming full part of the near-to real-time mine data flow, promising some interesting breakthrough in the quantitative rockmass response. Furthermore, combining seismic and geotechnical data analysis together with a 3D mine-data-based numerical model shows potential to build a more complete understanding of the rockmass response. The aim is finally to improve geohazard management in mines.

The I2Mine (Innovative Technologies and Concepts for the Intelligent Deep Mine of the Future) FP7 European project has been developed to propose activities designed to realize the concept of an invisible, zero-impact mine and to enhance the development of technologies suitable for deep mining activities. In this context, INERIS (National Institute for Industrial Environment and Risks) in partnership with the Swedish mining company Boliden, are currently implementing a new global monitoring approach in a new deep orebody, consisting in coupling both microseismic and stress monitoring observations with 3D modeling, as a challenging data integration solution to complete the geomechanical overview of the local (stress) and global (microseismic) dynamic mine response to the production. In this paper, the Garpenberg mine (Sweden) is introduced as well as the monitoring system that has been deployed, including acquisition, transmission process and data fusion together with production and geological mine data. The analysis focuses on a 10 months dataset. An interpretation is proposed, based on the associated dynamic response (microseismicity) and correlated with the mine data (geology, production volumes) through modeling. Finally, additional more recent stress shifts are considered with the perspective to complete the analysis and to optimize the fusion of 3D stress, microseismic and mine production data together with 3D modeling stress prediction capabilities.

2 Study area

The Garpenberg mine is located in the Dalarna County and it is the oldest mine still in activity in Sweden. In this mine, ore is nowadays extracted using sublevel stoping with paste filling and long-hole drilling method. It produces about 2.5 million tons of ore per year, composed of zinc, silver, copper, lead and gold and has now reached production depths higher than 1300 m below surface. Stope production is optimized by dividing the whole ore in primary and secondary stopes. Producing primary stope might significantly increase the stress level in the secondary stopes, which may softly yield or not, inducing seismicity and provoke instabilities or even rockbursts. Proposing good stope production sequencing is hence essential to minimize high-stressed zones. Indeed, as reported by Larsson (2004), in November 2003 the Garpenberg mine experienced a high seismic activity associated to a rockburst at 835 m depth, an event that was perceived on the surface as if it was a production blast. This event was caused by a failure of the sill pillar and provoked spallings in the roof and walls to a depth of about 25 cm, displacing about 1 ton of rock (Nyström 2003). Richter magnitude was estimated between -1.0 and 0.0 from Hudyma & Potvin (2004) classification.

In response to this event, a microseismic monitoring network was installed in the upper levels of the Lappberget area (up to approximately 1100 m depth) by the Institute of Mine Seismology (IMS) (Froehlich, 2014). However, the current exploitation of the mine (Fig. 1) is reaching levels as deep as 1300 m for which the IMS network coverage is not appropriated. To this purpose, within the framework of the I2Mine
project, it has been proposed to set up a new experimental stress and microseismic monitoring network in the deepest area of the mine, under INERIS supervision.

Figure 1  3D view of Lappberget monitored section with successive stope production in yellow, red, purple and blue for the 4 stress shifts that are analyzed in the present study and in grey for the other productions. CSIRO cells and microseismic 1C and 3C probes are also indicated as well as the primary stopes number (from 9 to 19)

3  Stress and microseismic monitoring

3.1  Monitoring network setup

Stress and microseismic sensors have been installed in a permanent way between December 2014 and February 2015 (Fig. 1). They were respectively pasted (with epoxy glue for CSIRO HI cells, well tested and documented in literature by Lahaie 2010) or grouted into horizontal, inclined, or vertical drilled holes. The location of these sensors was designed to guarantee significant stress changes in an area of interest considering the mine exploitation planning to come.

More specifically, the stress measurement layout consists of two stress cells installed at level 1157 (corresponding to the depth in meters) close to stope 13, planned to be mined in 2015-2016 (Fig. 1). PD CSIRO HI cell is set up in a downward borehole in stope 13 (primary stope) in the last section planned to be mined. In this way the monitoring network aims to assess the expected increase of the stress level in response to the progression of the nearby exploitation in a priori elastic conditions. PH CSIRO HI cell is set up in a horizontal hole in stope 14 (secondary stope) to compare the stress shifts between primary and secondary stopes until the end of the exploitation of the entire stope and maybe to reach the occurrence of failure. Each cell performs 12-strain measurements each hour with accuracy about ± 5µstrains, allowing stress shift determination by inversion. These measurements provide very local but accurate information to be compared with 3D numerical models and help for their calibration.

This instrumentation was preceded by a stress measurement campaign with overcoring methods (Amadei and Stephansson 1997) using also CSIRO HI cells located in the same borehole as for PH permanent cell, in order to assess the initial stress state (i.e. before any mining activity) and the relevant elastic parameters of
the rockmass. Indeed, this protocol allows to monitor the complete 3D stress path departing from the pre-existing stresses calculated in Table 1. The measurements obtained from overcoring are close each other in amplitude; however, a significant discrepancy, probably due to the stronger influence of the drift in H1, is observed. Global stress state from measurement is consequently calculated on the basis of H2 as a reference, less influenced by the drifts and closer to PH. Pre-existing stresses are also used as one of the most important input for model calibration.

### Table 1 Measured pre-existing stress field calculated from the overcoring samples with azimuth and dip angles

<table>
<thead>
<tr>
<th>Measurement index</th>
<th>$\sigma_1$(MPa)</th>
<th>Az.(°N)</th>
<th>Dip (°)</th>
<th>$\sigma_2$(MPa)</th>
<th>Az.(°N)</th>
<th>Dip (°)</th>
<th>$\sigma_3$(MPa)</th>
<th>Az.(°N)</th>
<th>Dip (°)</th>
</tr>
</thead>
<tbody>
<tr>
<td>H1</td>
<td>54</td>
<td>40</td>
<td>9</td>
<td>38</td>
<td>310</td>
<td>2</td>
<td>25</td>
<td>210</td>
<td>81</td>
</tr>
<tr>
<td>H2</td>
<td>54</td>
<td>0</td>
<td>7</td>
<td>44</td>
<td>91</td>
<td>5</td>
<td>26</td>
<td>219</td>
<td>81</td>
</tr>
</tbody>
</table>

In addition, six 1-component (1C) and five 3-component (3C) microseismic probes, equipped with 14-Hz geophones, have been deployed over the area of interest extending from reference levels 1108 to 1257 with an average inter-distance of 150 m. Seismic signals are acquired following an automatic advanced triggering scheme with an 8 kHz-sampling frequency. Once detected, waveforms are transmitted through the Local Area Network for automatic association. The microseismic events are then located using an inversion-routine based on the Oct-Tree method (Lomax et al. 2000). It consists in identifying the best position of the seismic signals source over a meshed search grid, using P and S waves’ arrival times and primary waves’ polarization angles. Indeed, it has been proved that the localization of a seismic signal with few probes can be improved with the integration of the polarization angles (Magota et al. 1987; Volker and Roth 2003). This approach has already been successfully applied in mine environments (Abdul-Wahed et al. 2001, Contrucci et al. 2010). Calculated P- and S-wave velocities have been estimated from five 0.5-to-2 Kg-load calibration blasts ($V_p$=6575 m s$^{-1}$ and $V_s$=3703 m s$^{-1}$). They showed good agreement with the expected values for such geological context (Zinszner and Pellerin 2007; Pasquet 2014); and their ratio ($V_p/V_s = 1.78$) and the equivalent elastodynamic parameters ($E = 70$ GPa and $v = 0.27$) are coherent. In such set up context, location error can reach up to 25 m, or higher depending on the Signal-to-Noise Ratio respect to the high mine activity (Matrullo et al. 2015). Finally, the relationship to estimate the local magnitude $M_s$ of these microseismic events is adjusted to fit IMS $M_s$ estimates that focuses essentially on the Kasperboo mining area, approximately located over Lappberget area.

### 3.2 Data transfer

Data acquisition is ensured by three SYTGEM Acquisition Units (AU), situated at levels 1108, 1182 and 1257. The SYTGEM technology offers both geotechnical and microseismic monitoring capabilities, featuring smart protocols to couple the two types of monitoring: AUs are interconnected and linked to a supervising PC on surface, from which communication and data transfer to the processing laboratories are managed through a specialized remote processing and database infrastructure so-called e.cenaris. In the present study, both monitored data and mining production information (blasts location and time, produced volume and surrounding geology…) are collected and transferred, enabling to quickly and remotely access an overview of the rock mass response (stress, microseismic) versus mining activity.

### 4 Data analysis

#### 4.1 Stress change measurements

In this study, three main strain shifts have been clearly measured between April and July 2015 through the stress cells at level 1157, plus an additional one in August 2015 at level 1207. These strain shifts were related to four important stope production blasts in the vicinity of the cells (up to 45 m between the...
excavation front and the cell location) as reported in Table 2. This table describes the main exploitation parameters, the distance of the excavated stope front to the CSIRO HI cells and the value of the most important stress change calculated after a given excavation (no indication of direction). The other stope excavations, considering their typical dimensions, were too far to be detected or with a very low signal so that inversion processing had no meaning.

Table 2   Details of the stope production blasts that were identified by the stress cells (positive value represent unloading)

<table>
<thead>
<tr>
<th>Blast day</th>
<th>Depth (m)</th>
<th>Stope #Step</th>
<th>Load (kg)</th>
<th>Distance stope front - cells PH</th>
<th>PD (m)</th>
<th>Assessed max. stress shift Δσ PH</th>
<th>PD (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>10/04/2015</td>
<td>1157</td>
<td>13#1</td>
<td>3553</td>
<td>35</td>
<td>35</td>
<td>8.8</td>
<td>2.4</td>
</tr>
<tr>
<td>08/07/2015</td>
<td>1157</td>
<td>13#2</td>
<td>4212</td>
<td>23</td>
<td>22</td>
<td>-1.0</td>
<td>-1.3</td>
</tr>
<tr>
<td>28/07/2015</td>
<td>1157</td>
<td>13#3</td>
<td>6669</td>
<td>5</td>
<td>8</td>
<td>-14.0</td>
<td>-8.4</td>
</tr>
<tr>
<td>11/08/2015</td>
<td>1207</td>
<td>13#1</td>
<td>4172</td>
<td>40</td>
<td>42</td>
<td>4.1</td>
<td>1.0</td>
</tr>
</tbody>
</table>

The strains recorded by the CSIRO HI cells after the blast (1157 13#1, 10/04/2015) present two features (Fig.2): firstly, the strain shift is unexpectedly important considering the mined volume (~1300 m³) and the distance between excavation front and CSIRO HI cells ~35 m (note the difference between PD and PH measurements which is due to excavation geometry: PH cell is directly oriented toward the produced stope while PD cell is in a “grey area”, as illustrated in Fig.1); secondly, additional strains are observed during several days after the blast (up to 1 month). The six-component stress shift tensor [Δσ] is assessed from the strains recorded by inversion calculation (Fig. 3 from PH measurements). The stress shift obtained immediately after the blast is much lower (5.6 MPa for the maximum amplitude) than the stress shift calculated on the basis of the cumulated strains until 10 days after the blast (8.8 MPa).

Figure 2   PD and PH strains recorded CSIRO HI cells (from 01/04/2015 to 30/04/2015)

On the contrary, the second blast performed at the same level (1157 13#2, 08/07/2015) is marked by a very low stress shift although the mined volume was much important (~2800 m³) and the distance from the front of the excavation to the stress monitoring CSIRO HI cells was lower (~22 m). The third and last blast at this level (1157 13#3) completed the exploitation of the remaining volume of this trench. It corresponds to an important excavated volume (~6000 m³) that was mined very close to the stress monitoring cells. The recorded strains are important with the emergence of plastic deformations at PH. In these conditions, the hypothesis of an elastic model is not valid anymore, which as a consequence limits the accuracy for the assessment of successive stress shifts.

The additional strain shift recorded after blast 1207 13#1 corresponds to the beginning of the exploitation of stope 13 at a lower level. The strains recorded are also important compared to the mined volume and the distance of the front to the stress monitoring CSIRO HI cells, even if these features are less significant than for 1157 13#1 blast.
Figure 3  Stress shift assessments (in MPa) calculated with the strain shift recorded at the stress monitoring CSIRO HI cell PH after blast 1157 13#1 (10/04/2015). Left: shift between states immediately before and after the blast; right: shift between states immediately before the blast and 10 days after the blast. The indicated North corresponds to the predefined North of the mine (along Y axis on Fig. 1)

Such observations proved the responsiveness of the stress cells to the close mine production. To get a better understanding of the unexpected amplitudes of the 3D stress shifts, investigations are proposed further from microseismic data analysis and 3D modeling over the same areas and periods.

4.2 Microseismic activity

Quasi static stress monitoring yields information on the mining-induced stress field, wherever stress cells have been judiciously implemented to monitor the 3D stress path before any failure occurs. Seismic monitoring instead provides information about rocks at failure, i.e. wherever stresses have exceeded the rock strength locally. It has thus been proposed to analyze the microseismic events that were detected in the same periods and located in a radius of 25 m around each blast location (approximately 3-stope width) to identify possible correlations between the production, the stress shifts and the microseismic response. The cumulated number of microseismic events has hence been plotted per “post-blast period”, where a post-blast period starts the day of the blast and finishes one day before the next of the four blasts (Fig. 4). For the last blast, it has been arbitrarily decided to finish the period one month after the day of the blast (i.e. the 11/09/2015). The time interval between each selected blast is varying but the first objective of such illustration was to quantify possible cluster of microseismic events associated in the space and time domains with the production areas. Although the mining process appears geometrically quite repetitive from one step to another, the microseismic responsiveness to a mining blast is not regular: blasts of the 10/04/2015 and of the 28/07/2015 seem to be followed by local microseismic events during several days, while nearly no seismicity is observed after the blasts of the 08/07/2015 and of the 11/08/2015 or scarcely during the first day. Then, while most of the microseismic events after stope blasts of the 28/07/2015 are detected the same day and in higher quantity, it takes much longer for the blasts of the 10/04/2015. This approximately 1-week response corresponds to the stress stability-return period (Fig. 2), which could be considered as a time dependent de-stressing phase. Such period length might be associated with the local geology and ambient stress level before blasting. This is successively checked with modeling analysis.
3D numerical modeling remains the only practical mean to assess the total stress field features related to mining sequences and stability issues. It calls for cautiousness due to the inherent limitations of the technology, for which a very simplified description of the rock mass is considered compared to its real complexity (void geometries and contrasted geology). Moreover, the dimensions of the incremental mining steps are small compared to the mining area to be modeled. This requires the meshing to be fine enough to ensure a good calibration of the mining-induced stress field. Here both stress and seismic field monitoring becomes prominent to assess how relevant the computed stress field is and to recall how much a good knowledge of the pre-existing principal stresses is crucial to compute correct solutions of the mining-induced stress field at any stage of the mine development.

3D numerical modeling therefore starts with the determination of the initial stress field, before any mining activity at all. To this purpose, it is necessary to model all excavations phases that were carried out before the CSIRO cells were installed. A 3D numerical model is designed with Flac3D software (Itasca 2012), in which initial stresses are adjusted to make the computed induced stresses compatible with the pre-existing ones measured by the CSIRO cells, i.e. in a zone already influenced by previous mining overlying levels. This knowledge is all the more important, since all the common parameters (stresses, safety factor, plasticity, stored elastic energy, strains) will derive from the initial stress conditions.

The model must extend from the surface up to a sufficient depth for the model boundaries not to introduce significant artifacts. Concerning horizontal extensions, to limit the model size, the influence zone of the stress measurement is assumed to be over maximum 3 stopes on each side. Finally, the resulting 3D model has a volume of 2.7 km$^3$ over a 1500 m height and is composed of 10 600 000 hexahedral zones. One of the difficulties in this modeling is to deal with the simplified description of the ore body and weaknesses zones (mainly composed of talc that locally influences the stress field, composed of sericitic schist (soft) to talcy schist (very soft)). The chosen rheology for these materials is an elastoplastic law with a parabolic failure criterion (Hoek & Brown, 1997) and the geomechanical parameters needed in the model are determined from lab testing (Table 3).
The stress calibration is processed by assuming test values in the model for the 3 principal initial stresses before any excavation happened; then the upper mining levels (from 578 m to 1088 m depth) are excavated and backfilled, and finally, the few recent drifts and the stopes excavated in the zone of influence of the field measurements before December 2014 are deleted in the model. In the present study, 2x3 initial stresses, which correspond to 2x6 total induced stresses ($\sigma_{xx}, \sigma_{yy}, \sigma_{zz}, \sigma_{xy}, \sigma_{xz}, \sigma_{yz}$) have been computed at two different locations (H1 and H2, close to PH permanent cell, Fig. 1). When those computations are repeated sufficiently (computed stress data bank), it is possible to back-compute the stress states in those 2 locations that minimize the difference between measured and computed stresses. The minimization (Levenberg-Marquardt method, Levenberg 1944) is done on the following 3 $f$ and 6 $g$ order 2 polynomial functions ($f_i$ & $g_i$): $\sigma_{xx}(i) = f_i(\sigma^{\text{induced}})$, $\sigma_{yy}(i) = f_i(\sigma^{\text{induced}})$, $\sigma_{zz}(i) = f_i(\sigma^{\text{induced}})$ and $\sigma_{xy}(i) = g_i(\sigma_{xx}(i), \sigma_{yy}(i), \sigma_{zz}(i))$ (i: 1 to 6). At the end, the best solution for both locations is determined when the domain of initial stresses is compatible with the confidence interval of measured stresses and densities (intersection of regions created with fitting functions $g_i$):

$$\sigma_{\text{min}} < g_i(\sigma_{xx}(i), \sigma_{yy}(i), \sigma_{zz}(i)) < \sigma_{\text{max}} (i: 1 \text{ to } 6)$$

$$\rho_{\text{min}} < \sigma_{xx}(i) < \rho_{\text{max}}$$

For Lappberget section at -1155 m level, the retained computed initial stress state is:

$$\sigma_{xx}^{\text{ini}} = 44.3 \text{ MPa}, \sigma_{yy}^{\text{ini}} = 47.3 \text{ MPa} \text{ and } \sigma_{zz}^{\text{ini}} = 3030 \text{ kg/m}^2 \text{ g} \times 1155 \text{ m} = 34.3 \text{ MPa.}$$

With these initial values, the maximal difference between the theoretical stress values and the values extracted from overcoring (Table 1) is of 2.5 MPa.

### 5.2 Field stress monitoring

The CSIRO cells permanently installed in locations PH and PD (Fig. 1) record stress shifts due to any significant excavation not far away. Theoretically, an excavation induces a stress shift proportional to the excavated volume and inversely proportional to the third power of the distance. To compare predicted stresses from numerical modeling to the measurements obtained with the permanent cells, a 12-stage phasing (gallery and stope excavations) has been carried out with the Flac3D model. At each stage, the stress components of the previous stage are subtracted to those of the current stage, exactly as stresses are measured by CSIRO cells. For the three strain shifts at level 1157, the comparison between modeled...
and measured stress shift (for the 3 principal stresses) can be analyzed from Fig. 5 and 6. A significant difference is noted for the excavation of 10/04/2015 (blast of 3553 Kg explosive for an approximate volume of 1300 m$^3$ of ore): maximum measured stress shift is 1.8 times greater than the maximum modeled stress shift. This is due to the presence of the weakness body: its geometry is evaluated by mine’s geologist and then simplified in the model, which can lead to significant differences with the reality. Indeed, without taking the weakness body into account, the measured stress shift is 3 times lower than the modeled one. This proves the strong influence of much contrasted units neighboring the mining voids, inducing much higher stress transfer on the hardest rocks. On 28/07/2015, a bigger (2800 m$^3$) closer (22 m far from the cells) excavation occurred, which produced a lower stress shift than the excavation of 10/04/2015 (1300 m$^3$ at 35 m) according to the 3D model. This observation confirms the assumption that the presence of a weakness body (expected to be talc) may strongly impact the stress field. For both excavations, the cells located at both PH and PD measured the same trend as modeling: all principal stresses have the same sign (- for loading or + for unloading). Moreover, the analysis of the closer excavation (28/07/2015) shows that the modeled and measured stress shifts have different amplitudes for 2 components. The numerical model obviously overestimates the major principal stress and thus the stored strain energy. This can be explained by at least two reasons: 1) the CSIRO cells are in a damaged zone where stresses are not stabilized yet; 2) the simplified geometry of the real excavated stope is not accurate enough, as well as the influence of the surrounding geological discontinuities. One can suspect that modeling the paste-filling could reduce these differences.

Figure 5  Measured and modeled stress shifts (smax = major, sint = intermediate, smin = minor) induced by stoping development at PH between April and August 2015. Information on the excavated volume (V), the distance from the cells (dist.) and the cumulated radiated seismic energy (purple circle) are added
Figure 6  Measured and modeled stress shifts (smax = major, sint = intermediary, smin = minor) induced by stoping development at PD between April and August 2015. Information on the excavated volume (V), the distance from the cells (dist.) and the cumulated radiated seismic energy (purple circle) are added

As a conclusion, besides all possible refinements, from the determination of the initial stress state and the following readjustments with the recorded strain shifts, it is shown that a 3D model can most likely be calibrated on permanent stress monitoring data. Such continuous update enables to study the impact of different mining scenarios on a local area of high interest. A second potential perspective is also the computation of stress paths into a database to allow a comparison between field and predicted values on a routine basis to potentially issue an alert. This requires high-quality mine data flow in the space-time domain to remove significant artifacts in numerical modeling.

5.3 Modeling perspectives

Table 4 lists the various numerical indexes suitable for modeling continuous media. Some of those indexes are classified in rock burst categories (proneness from none to strong rockburst, Zhou et al. 2012). The main future development of this numerical model will consist, besides the mining induced stress field, in computing the most appropriate criterion for mapping seismic hazard proneness.

Table 4  Criteria and indexes of rock burst proneness computable in a numerical model

<table>
<thead>
<tr>
<th>Reference</th>
<th>Index</th>
<th>Equations</th>
</tr>
</thead>
<tbody>
<tr>
<td>Tao (1988)</td>
<td>Activity index</td>
<td>(\sigma_1/\sigma_c)</td>
</tr>
<tr>
<td>Turchaninov et al. (1972)</td>
<td>Turchaninov criterion</td>
<td>((\sigma_0 + \sigma_1)/\sigma_c)</td>
</tr>
<tr>
<td>Barton (Tang, 2000)</td>
<td>Criterion of tangential stress</td>
<td>(\sigma_0/\sigma_c)</td>
</tr>
<tr>
<td>Hoek &amp; Brown (1997), Wang et al. (1998)</td>
<td>Criterion of tangential stress</td>
<td>(\sigma_0/\sigma_c)</td>
</tr>
<tr>
<td>Jiang et al. (2010)</td>
<td>Local energy release rate</td>
<td>LERR = (W_{e\max} - W_{e\min})</td>
</tr>
<tr>
<td>Wiles (2002)</td>
<td>Loading System Stiffness</td>
<td>LSS = (p/\varepsilon_v)</td>
</tr>
<tr>
<td>Cook (1966), Stacey &amp; Page (1986)</td>
<td>Energy release rate</td>
<td>ERR = ((W_e + W_i)/\text{Volume})</td>
</tr>
<tr>
<td>Tadjus et al. (1997)</td>
<td>Coefficient of energy concentration</td>
<td>(\beta = W_e/W_e^i)</td>
</tr>
<tr>
<td>Tadjus et al. (1997)</td>
<td>Coefficient of vertical stress</td>
<td>(\sigma_{zz}/\sigma_{zz}^0)</td>
</tr>
</tbody>
</table>
6 Merging analysis

Stress CSIRO HI cells recorded an immediate significant stress shift on 10/04/2015 with successive time-differed stress shifts during several days that were associated with microseismicity over a whole month, while the following production of 08/07/2015, although much larger, revealed a smaller stress shift with nearly no seismicity. These observations were investigated and confirmed by the results from the modeling of a large weakness zone. Indeed, weakness zones (such as talc levels) do affect wider areas by diffusing a slower kinetic plasticity. As a consequence, the influence area of the excavated volumes is larger. This induces ductile behavior inside the weakness zones and fracturing in the nearby hard rocks (i.e. seismicity) for the stress regime to stabilize over several weeks. In this way, once next stope section is excavated, no significant seismicity or stress shift are observed since the stope has already progressively been distressed.

This behavior was observed for the important stress shift of 11/08/2015 too where seismicity was detected in the same area over 2 weeks; and more recently after first stope production at level 1182 on 26/02/2016, also located close to the weakness area. Such observations enable to confirm the progressive propagation of the stresses inside the ore, for sections containing weakness areas or not, with possible consequences of the stress transfer from the primary to the secondary stopes.

Finally, additional analysis, e.g. stress inversion calculations and accurate modeling based on mine production dataflow (geology, approximate produced ore dimensions...), possibly on new strain-shift-inducing production blasts, followed by microseismicity or not, are required to propose a longer-term valuable interpretation of such behavior.

7 Conclusion

An innovative global monitoring strategy is currently being implemented in the deeper levels of the Garpenberg mine (Lappberget orebody), where sublevel stoping method generates complex mining-induced stress fields. Induced stress fields might represent an issue to select the best mining sequence versus ground failure and seismic hazard assessment. In this monitoring, technological and geological mine data flow are being considered on a routine basis as a primary source of information to quantify and analyze the mining-induced stress state and the seismic activity. Coupling field stress and seismic monitoring along with stress prediction from 3D modeling is also part of this four-fold monitoring strategy.

During the first year of monitoring, stress changes associated with seismicity in the monitored rock mass have shown preliminary results of promising interest. First is that mining-induced seismicity and stress change rates in the surrounding rock of the freshly excavated volume appear to be closely related. Especially, important relaxation of the surrounding rock is observed when the excavation is in the limits of a large ductile area. This is most likely induced by time-dependent deformation and asymmetric stress transfer onto the hard rock medium. In this case microseismic response is staggered.

3D numerical modeling of the complete orebody including past mining levels has been calibrated based on mining-influenced stress field measurements. Initial stress field have been retrieved thanks to a computing inversion procedure that offers an excellent way to consider the stress field measurements even if acquired in an area already disturbed by the production.

Refining the geology in the model significantly reduces the discrepancy between stress change measurements and prediction, recalling here the weight of an oversimplification of the geology in mine-scale numerical simulations. This result underlines the mechanism of a time-dependent stress field arching preferentially on the hard rock medium, a phenomenon suspected of favoring dynamic instabilities and already encountered from the past experience in the mine. Shortcomings in the 3D numerical simulations could be overcame by implementing additional stress cells and geomechanical sensors in areas expected to be exploited and likely to be under the influence of weakness area.

As a further development to refine the 3D model, the choice of the most adapted criterion (numerical index) for assessing the rock burst proneness should be based on the correlation with seismic events.
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References


Investigations of hard coal mine roadways stability in stratified rock

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Abstract

300-350 kilometers of development roadways are driven in Polish hard coal mines yearly. They are predominantly arch shaped with dimensions ranging from 4.0-8.0 m in width, 3.5-5.0 m in height and are located at depths of 500 m-1300 m. The variety of roadway size, depth, location (in the fault zones) and differentiation of geologic and mining conditions determines the type of support being used. Observational studies of rock mass behavior around the workings indicate that roadway stability depends on the degree of stratification in the overlying rocks.

These observations have been carried out in selected coalmine roadways with varying mining and geological conditions. In the course of research, range and intensity of the fracture zone was studied by the use of an endoscope and extensometers, as well as loads on the support with the help of instrumented bolts. Convergence monitoring was performed. The in situ research is related to the lithology and geomechanical parameters of the rock mass tested in the laboratory and with the help of a penetrometer. Duration of the research cycles varied from a few to several dozen months for every roadway.

The results of studies presented in this paper take into consideration the lithology and roof rock stratification influence on roadway stability. Support schemes have been considered as well. Results of the analysis show that rock mass lithology has significant impact on measured quantities and, in effect, on roadway stability. This is dependent on the variable roof rock thickness rather than its compressive strength. In case of roadways with thin claystone and mudstone beds in the roof, the number of fractures and range of the damage zone is higher (usually a few meters) compared to sandstone rock mass. Sandstone rocks are more monolithic and have higher thickness, which results in minimal rock separation and small range of the roof fracture zone. The above observations can serve as a basis for verification of the designed support and can help in selection of the proper numerical or analytical model, which is essential to the mining design processes.

1 Introduction

About 300-350 km of roadways are driven in Polish coal mines each year. The depth of mining works ranges from 500 m up to 1,300 m, so geological conditions at the sites vary a lot. These conditions include rock parameters, their inclination, bed thickness, water conditions and faults as well as other geological disturbances (Małkowski et al. 2012). The above factors affect the support design and its behavior even more than the mining and technological factors (e.g. other workings, working size or time range of working maintenance). It is worth noting, however, that mining factors are affected by multi-seam exploitation, which creates very complex stress and strain situation in the rock mass.

Several coal seams are sometimes mined in one 50-70 meter vertical section of the rock mass. Left pillars and exploitation edges interact on running mining works in adjacent area, which effects stress concentration and mining tremors. At the same time, rock mass properties continue to be the key to
preventing potential rock failures and maintaining roadway stability (Bukowska 2005), especially considering that steel yielding arch support is the most common type of support used in Polish hard coal mines.

One of the most important features of carboniferous rock formation is the stratification of thin claystone, mudstone, coal and sandstone beds. This is especially true of claystones and coal beds of minimal thickness, often below 1 cm. Geological research in the Silesia region of Poland proved that the average thickness of typical rock beds is 41.1 cm for sandstones, 28.7 cm for mudstones, 20.7 cm for claystones and 34.0 cm for coal (Dubiński & Konopko 2000). Additionally the coal has very distinct cleavage. Rock mass bedding has a remarkable influence on rock mass behaviour around the roadway (Brady & Brown 2006, Małkowski 2015). However, it is rarely considered in geotechnical classifications. Although discontinuous spacing, joint set number, spacing intensity, volumetric joint count or rock mass structure can be found in the most widely used classifications as: GSI, RMR, Q, CMRR or RMi systems, there is no information about the lithology or rock mass stratification (Bieniawski 1989, Hoek 2007, Barton 2002, Mark & Molinda, 2003, Palmström 2000).

Underground research has been carried out to assess the effect of lithology and roof rock stratification on roadway design and maintenance. Research was conducted in selected coalmine roadways located in different mines with different mining and geological conditions. To this end, measurement stations were built. Length of the research cycles varied from a few to several dozen months for every roadway. Roadway stability was assessed by observing roof rock fracture zones, roadway convergence as well as loads on the steel yield arch support. Selected results of the studies are presented in this paper.

2 In situ research

The authors of this paper have been carrying out underground investigations for many years. They were conducted under various geological and mining conditions, which enabled the authors to select cases with varying roof conditions. Some of them featured the same mining conditions around the roadways and similar depth of their drivage, but with a different set of beds and their properties. The three different types of rock mass presented in the paper are as follows:

- massive rock mass - poorly bedded,
- blocky rock mass - medium bedded,
- laminated rock mass - strongly bedded.

First, 7-8 m boreholes were drilled in the roof in order to evaluate lithology and geomechanical parameters of rocks. The fracture (damage) zone ranges were determined with the help of borehole endoscope (infra-red camera), separation was measured with the help of magnetic extensometers, convergence based on surveying measurements and loads on the arch support with the help of hydraulic dynamometers (Majcherczyk et al. 2008, Bigby et al. 2010, Niedbalski et. al. 2013, Spearing et al. 2013). Typical steel yielding arch support, in this case reinforced with bolts, is shown in Figure 1.
2.1 Geological conditions at the sites

In order to assess support work and behavior of a rock mass around the workings, investigations have been conducted in three roadways in two different mines:

- belt incline C-3 in seam 502/1 at depth 820 m – in a massive rock mass - poorly bedded,
- roadway Cz-6a in seam 358/1 at depth 880 m – in a blocky rock mass - medium bedded,
- roadway B-3 in seam 358/1 at depth 930 m - in a laminated rock mass - strongly bedded.

The roof lithology description is based on core logs drilled to the depth of 7 m (Fig. 2). The roof of belt incline C-3 consisted of a single sandstone bed (with varied grain size). All carboniferous beds: claystone, mudstone and sandstone, were present in the roof of roadway Cz-6a (seven beds) and roadway B-3 featured ten beds of claystone and sandstone, sometimes with coal laminas. The floor of discussed drifts was built of thin layers of claystones and mudstones.

![Figure 2 Lithology in the roof of: a) Belt incline C-3, b) Roadway Cz-6a, c) Roadway B-3](image)

The laboratory tests performed on samples cut from the core logs and in situ tests conducted with the help of penetrometer allowed to determine geomechanical parameters gathered in Table 1. The highest compressive strength of 88 MPa was found in rocks of the most bedded roof of roadway B-5,
while the lowest 45 MPa was found in rocks in the most intact roof of belt incline C-3. The RQD index value confirms that the quality of roof rocks in roadway B-5 is the poorest among all tested ones (RQD = 40.3%) due to its intensive bedding. The Rock Quality Designation of monolithic sandstone in the roof of belt incline C-3 is 84.4%. The RQD was lower for the rocks in the roof of the remaining ways, amounting to 40 ÷ 57.9%. This can be attributed to greater stratification of their roofs, despite the significantly higher compressive strength noted in belt incline C-3.

**Table 1  Results of rock investigations**

<table>
<thead>
<tr>
<th>Working</th>
<th>( \sigma_{c\text{lab}} ) [MPa]</th>
<th>( \sigma_{t\text{lab}} ) [MPa]</th>
<th>( \rho ) [kg/m(^3)]</th>
<th>RQD</th>
<th>( \sigma_{c\text{pen}} ) [MPa]</th>
<th>( \sigma_{t\text{pen}} ) [MPa]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Belt incline C-3</td>
<td>45.26</td>
<td>5.29</td>
<td>2409.3</td>
<td>84.4</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Roadway Cz-6a</td>
<td>73.20</td>
<td>6.65</td>
<td>-</td>
<td>57.9</td>
<td>40.46</td>
<td>3.23</td>
</tr>
<tr>
<td>Roadway B-3</td>
<td>87.97</td>
<td>5.56</td>
<td>2630.1</td>
<td>40.3</td>
<td>55.74</td>
<td>5.11</td>
</tr>
</tbody>
</table>

Aside from the geological conditions, it should be noted that rock mass around the analyzed workings had a slope of 5 ÷ 10 and that the workings were not in the range of influence of any tectonic disturbances or subject to changed mining conditions during the study period.

The applied supports consisted in the following elements:

- belt incline C-3 - yielding steel frame with a width of 5.0 m and a height of 4.02 m built of V29 sections spaced at 1.0 m,
- roadway Cz-6a – yielding steel frame with a width of 5.0 m and a height of 4.02 m built of V29 sections spaced at 1.2 m, with the roof additionally reinforced by two steel bolts with a total length of 2.5 m, fixed to the every roof bar,
- roadway B-3 in seam 358/1 – yielding steel frame with a width of 5.0 m and a height of 4.02 m built of V29 sections spaced at 0.75 m.

### 2.2 Convergence measurements

For the purpose of evaluating the operation of supports in the selected workings, changes in roadway width and height were subject to measurement. For belt incline C-3, measurements were carried out at four positions for a period of 750 days, for roadway Cz-6a, measurements were carried out at three positions for a period of 156 days, and for roadway B-3 measurements were carried out at three positions for a period of 477 days. Due to technological reasons, convergence measuring stations have been built behind the roadway face. The stations at belt incline C-3 63 days after its driving, at roadway Cz-6a – 50 days and at roadway B-3 – 38 days later. Fig. 3 shows the average convergence values for all positions in each working. Research shows that the greatest value of convergence recorded was -68mm, resulting from the change in the height of roadway Cz-6a. Despite over two years of continuous observation, the slightest change was observed in belt incline C-3, with final horizontal and vertical compression value of less than -30 mm. These values show that despite its low compressive strength, the presence of sandstone in the roof and the floor of a working promotes lower convergence values. All the measured convergence values in the selected workings do not threaten their stability, but in the long term the surrounding rock mass will have a greater tendency to move towards the selected area.
2.3 Fracture zone measurements in the roof

The phenomena occurring in the roofs of workings were measured using sonic extensometers and tell-tales to evaluate displacement of the roof layers. Extensometers were used to record changes in roof layer displacement for belt incline 3-C (Fig. 4) and roadway B-3 (Fig. 6). Displacement in roadway Cz-6a was determined by tell-tales (Fig. 5).

Positive values indicate displacement with respect to the base point, while negative values indicate compaction. In the case of steel frame supports, the layers are supported by frames, which may result in compaction of layers located directly above the roadway (until frame yields), or those with lower stiffness. This phenomenon was observed in belt incline C-3 and roadway B-3. It’s worth noting that roof strata movements were inconsiderable for the first 50-60 days, afterwards they started to increase. For belt incline C-3, which is located in poorly bedded massive rock mass, roof layer displacement value was in the range of -15 -10 mm (Fig. 4). In general, it can be said that the movement of roof layers in the analyzed roadway was minor, but the roof strata division for packages which move separately was very clear. Some of them move up (e.g. anchors at the depth 0.31, 0.62, 1.00 m) and some move down (e.g. anchors at the depth 1.31, 1.62, 2.05 m) periodically. It can happen when the roof rocks are not bolted. The change of mining situation in the rock mass even with a distance of several meters from the roadway is the reason. It should be underlined that the sudden roof displacement is only a part of vertical convergence which comprises also with elastic rock deformations and floor heaving.

Figure 3 Convergence changes in time for the analyzed workings
Investigations of hard coal mine roadways stability in stratified rock

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Figure 4 Displacement of roof layers for belt incline C-3 on the basis of extensometric measurements

For the medium bedded rock mass (roadway Cz-6a), studies was carried out over the period of four months at three tell-tales located at the depth of 2.5 m in the roof (“low tell-tale” - RN) and three tell-tales located at the depth of 5.5 m in the roof (“high tell-tale” - RW). Roof beds separation shows continuous growth there (Fig. 5). The lowest value of -21 mm was recorded at RN2 (for 2.5 m section of roof rock), while the highest value was found in RW3 (for 5.5 m section of roof rock). Due to the short period of observation (4.5 months), it is impossible to say if the rock mass movements in the roof have stabilized. It should be underlined that rock bolts used in the roof changed the rock mass separation character, which increased continuously, what indicates the rocks movement towards the roadway opening. The separation reveals just above bolted roof strata, because the values of separation for “low” and “high” tell-tales were very similar for their same position in the roadway.

Figure 5 Separation of roof layers in roadway Cz-6a

Compression of some layers located directly above the support is clearly observed in roadway B-3, where it occurs to a depth of approx. 2.6 m (Fig. 6). Full separation of the roof rock usually generated displacement in the range of -20 to 20 mm, which is the highest among chosen roadways. So the beds above a depth of 3.5 m separated and the lower section of the roof strata “seated” on the standing support frames revealed a compression.
The separation measurement results indicate that the roof of the working in the poorly bedded massive rock mass shows smaller changes than the other workings driven in medium and strongly bedded rock mass. The influence of a type support for the roof layers movements is visible at the same time. Non-bolted roof layers can move freely, but 30-40-millimeter roof rocks separation for roadways with steel yielding support doesn’t pose a risk of losing its stability.

Figure 6 Displacement of roof layers roadway B-3 on the basis of extensometric measurements

To assess the nature and extent of fractures in the roof area of all the workings, endoscopic measurements were carried out, consisting in direct observation of borehole walls.

For belt incline C-3, endoscopic examination was carried out in two roof boreholes: one vertical and one inclined. The base measurement performed in the face of the working immediately after the drilling of boreholes indicated no fractures. The next measurement in the vertical borehole showed only one fracture with opening size of 5 mm at a depth of 0.5 m. In case of the inclined borehole, no discontinuities were observed even after 26 months (Table 2). The reasons for this can be attributed to lithology, as the roof of this working consists of a solid sandstone blocks with considerable thickness and minor stratification. Thus, the displacement of roof layers found using an extensometer was elastic in nature.

Table 2 Summary of endoscopic measurement results for belt incline C-3

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Borehole T-117 – vertical</th>
<th>Borehole T-118 – inclined</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>base</td>
<td>after 12 months</td>
</tr>
<tr>
<td>Number of fractures</td>
<td>0</td>
<td>1</td>
</tr>
<tr>
<td>Total separation [mm]</td>
<td>0</td>
<td>5</td>
</tr>
<tr>
<td>Fracture zone range [m]</td>
<td>0</td>
<td>0.50</td>
</tr>
</tbody>
</table>
Endoscopic measurements performed in the roof of roadway Cz-6a shows the develop of the fracture zone (Fig. 7). There were 9 discontinuities with 19 mm separation up to the depth of 1.4 m immediately after opening the roadways face. Six months after the first measurement made in the face of the working, the number of fractures increased from 9 to 18, displacement increased from 19 to 45 mm and the fracture zone range from 1.4 m to 2.5 m (Fig. 7). Thus, a bedded roof layer was conducive to roof rock separation. The observed fractures differed in nature, consisting of cracks with no signs of separation, fractures and fractures filled with debris. The fracture separation corresponds with roadway Cz-6a vertical convergence.

![Measurement results](image)

**Figure 7  Endoscopic measurement results for roadway Cz-6a**

Endoscopic measurements of the fracture zone range and the amount of separation performed immediately after drilling a borehole in the roof of roadway B-3 (at the roadway face) showed 18 discontinuities with total displacement of 80 mm occur shortly after the roadway was driven, with the last discontinuity observed at a depth of 2.9 m (Table 3). The last measurement showed increase in all measured parameters, and thus the fracture opening size increased to 160 mm and the fracture zone range to 3.8 m. Thus the range of damage zone, number of fractures and their separation for the strongly bedded roof is the highest. The scheme of discontinuities in the roof is shown in Fig. 8 and sample images of endoscopic procedures are shown in Figure 9.
Endoscopic measurements clearly indicate that lithology and stratification of roof rocks influence the intensity of fractures in the roof layers located directly above the working (Fig. 9). The rocks lamination also makes them being prone to fracturing (Lubosik et al. 2015). The smallest fracture zone was found in belt incline C-3, which was driven in poorly bedded rock mass and the biggest fracture zone was found in roadway B-3 driven in strongly bedded rock mass.

![Figure 9](image.png)
2.4 Support load measurements

In assessing the impact of fracture zones on the supports, hydraulic dynamometers placed in the roof between the rock mass and the support frames or arches were used to measure the load carried by the support frame.

For belt incline C-3, the results showed a minor load value amounting to 15 kN under the right wall arch already during the first measurement, which was performed four days after the installation of dynamometers in the face of the working (Fig. 10). Further measurements carried out over a period of two years showed little change, with values in the range of 0-15 kN. The values obtained confirm minor impact of rock mass on the support for belt incline C-3.

**Figure 10** Loads on arch support for belt incline C-3

In the case of roadway Cz-6a, load increment was observed for a much longer period, because the reading carried out three days after installation of a dynamometer in the roof showed no load (Fig. 11). Subsequent measurements carried out once a month showed gradual increase of up to 60 kN load after approx. 100 days. The next measurement carried out after four months showed a load of 68 kN. Thus, it can be concluded that growing fracture zone present in the roof of the analyzed working caused a gradual increase in loads on the support frame.

**Figure 11** Loads on arch support for roadway Cz-6a
Support loads of a different nature were found in roadway B-3, where a load of 78 kN days was recorded after a period of 17 days (Fig. 12). Already in the initial stage, a significant fracture zone was observed in the roof of the working. After this period, there was a decrease in loads on the individual dynamometers, which shows the stress release. In the following period of time, load fluctuations occurred, with the final measurements showing an increase. It should also be noted that the support frame was loaded asymmetrically, as the loads recorded by dynamometers installed in the right and the left ribs showed different values. Taking into consideration that there is a rock plate above the roadway which is able to slightly move up and down periodically such a behavior is possible. Similar results got the scientists from Central Mining Institute in other coal mines (Walentek et al. 2013, Lubosik et al. 2015), where a load on the standing support also was dropping down regardless lower values than support load bearing capacity. The accuracy of a load measurement was ca. 3 kN.

![Figure 12 Loads on arch support for roadway B-3](image)

Overall, the support load values in the individual workings are derivative of lithology and bedding of the roof layers. Despite the lowest strength parameters and the longest period of measurement, belt incline C-3 showed the lowest load values. The rock mass susceptibility to separation results in an increase in loads on the support. However, the increase in loads also depends on the intensity of roof rock bedding.

3 Conclusions

Results of this study and authors experience indicate that the value of measured parameters, and thus the stability of roadways, is very much influenced by the lithological formation and stratification of roof rocks. The rock strength parameters are of little importance in this case, since the smallest changes were observed around belt incline C-3, even though laboratory compressive strength of rock layers was 40% lower than the in other workings. Solid sandstone rocks surrounding roadway C-3 displayed a tendency to form monolithic layers, which generated minor displacement with approx. 5 mm range and a slight fracture zone of up to 0.5 m. In the case of workings featuring claystone, rock layers showed high propensity for roof rock separation of up to 160 mm and a significant fracture zone range, i.e. 3.8 m.
Changes in the working environment affect the loads on the frame support construction. The load on the support frame in poorly bedded sandstone was 15 kN, while in the strongly bedded rock mass it reached up to 80 kN. It didn’t reach the load bearing support capacity in both cases, but obviously many thin rock beds in the roof separate more easy and load the standing support. The load on the standing support varied over 5 times in the investigated roadways.

Summing up the results of these measurements, it should be noted that rock layer movement in bedded rock mass surrounding the workings occurs in a continuous manner. One cannot unequivocally say that a certain period of time will be followed by a secondary equilibrium of the rock mass, which guarantees the absence of displacement. The greatest intensity of movement in the rock mass around the workings occurs in the first 2-3 months after driving the roadways in polish hard coal mines geological conditions. Roofbolting restrains slightly the roof rocks movements.

The main problem in any mine for adjusting the roof support system to the local mining and geological conditions is to get a quantitative assessment of surrounding rocks behaviour. Then you can establish the threshold values of roof rocks separation or fracture zone range which can be danger for the roadway stability. These values, in turn, depend on the support type. The results presented in the paper show that the approach to the roadway stability prediction with bedded roof should be different than to the solid monolithic rocks (e.g. metamorphic). Thus, roadways and rock mass stability monitoring is indispensable.

Acknowledgement
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Laboratory investigation of rock-shotcrete deboning due to ice growth using acoustic emission

G. Mainali, S. Dineva and E. Nordlund, Luleå University of Technology, Sweden

Abstract

Road and railway tunnels in cold regions are often affected by problems related to water leakage and freezing temperatures. Water leakage in a tunnel leads to ice growth when the temperature goes below freezing and creates favourable environment for fallouts of shotcrete and rock. This paper presents results and observations from laboratory freezing–thawing experiments on rock blocks covered with shotcrete and focuses on the degradation of the shotcrete-rock interface due to ice growth. The initiation and the development of freeze-induced micro cracks in shotcrete-rock interface were studied by continuously monitoring acoustic emissions (AE) and temperature. The clustering of the AE events during freezing and thawing indicates that micro cracks appeared in the shotcrete-rock interface and caused adhesion failure. The larger number of AE events in the panels, with access to water during freezing, confirmed that water contributes to material deterioration and also reduces the adhesive strength.

1 Introduction

Over the past twenty years, Swedish Transport Administration (STA) has observed an increased number of incidents involving shotcrete and rock fallouts. Several cases of rock and shotcrete fallouts in different tunnels in Sweden have been reported by Andrén (1995). Field observations were undertaken by Andrén (2008) in five Swedish railway tunnels. The results show that the water leaking into the tunnels begins to freeze and ice formations such as icicles and ice pillars form in the beginning of the winter. This causes major problems related to train operation, derailment, short-circuit and safety of working environment. A number of international studies have also shown that functional damage due to icing and frost damage is common in many countries (ITA, 1995; Lai et al., 1999; Zhang et al., 2004; Thomachot et al., 2005).

Shotcrete is commonly used as surface support in Swedish railway tunnels. There are only a few studies on debonding of rock-shotcrete due to frost action, e.g. Andrén (2009). This problem is of a great concern since it can cause fallouts of shotcrete and rock (Andrén, 2009). During the freezing period, cracking in rock and shotcrete may occur due to development of ice pressure in the interface that can reduce the overall strength and stiffness of the rock support. The result from a field survey of the temperature variations in railway tunnels in the northern Sweden showed that the temperature passed 0°C around 20 times during a single winter season (Andrén, 2008).

The micro cracking due to freeze and thaw generates elastic waves which travel through the material and can be recorded and analyzed. In several recent studies (e.g. Girard, et al. 2012 and Amitrano, et al. 2012), AE measurements were conducted to monitor freezing-induced damage in alpine rock-walls. Mainali et.al (2015) studied AE activities in shotcrete during freezing and thawing and found most of the AE events generated during freezing cycles.

This paper presents results and observations from laboratory freezing–thawing experiments on rock blocks covered with shotcrete and focuses on the degradation of the shotcrete and the shotcrete-rock interface. The freezing rate and water access conditions were varied among the tested samples.
Laboratory investigation of rock-shotcrete deboning due to ice growth using acoustic emission

G. Mainali, S. Dineva and E. Nordlund

2 | Ground Support 2016, Luleå, Sweden

AE activity - the number of events, source locations and size of the AE events induced by cracking in the rock-shotcrete interface during repeated freezing and thawing cycles are studied here.

2 Experimental Methods

2.1 Test panels

The bedrock in Sweden is dominated by hard crystalline rocks such as granite, gabbro, and schist. A thin shotcrete layer of about up to 50 mm is usually used for the rock support in the tunnels. In this study the specimens for the laboratory experiments were three panels of Kuru granite (fine-grained, equigranular and isotropic) with a 30 mm thick shotcrete layer to simulate the conditions in the tunnels. The panels had dimensions 800 x 800 x 50 mm. Eight holes with a diameter of 5 mm were drilled through the rock panel to supply water to the interface during the laboratory tests (Figure 1). After drilling the test panels were sandblasted and the surface around three of the drill holes were treated with geotextile, three of the holes with white wax crayon while the surface around the remaining two holes were left without preparation. The geotextile was used to mimic the conditions of fracture opening and fracture filling material which would allow the water to be supplied around the opening. The wax crayon was used to simulate the behaviour of rock surface with poor adhesion. The sandblasted surface was then covered with 30 mm of shotcrete.

![Figure 1 Plan view of the test panels with temperature sensor locations and drill hole location with different type of preparation (see the legend)](image)

2.2 Freeze-thaw tests

The damage development was studied by evaluating the intensity and the number of events, location and energy of AE during the 20 freeze-thaw cycles. The freeze-thaw tests were performed at Luleå
University of Technology in a cooling chamber with a constant temperature of +4 °C. The cooling system consisted of a cryostat (filled with water with 20-30% ethanol) that circulated in copper pipes at the bottom of the cooling box at a temperature around -15 to -10°C. A steel plate was placed on the top of the pipes to distribute the temperature evenly across the test panel. A layer of sand was placed on the plate to level the test panel in the cooling box (Figure 2). To ensure identical freezing and thawing cycles for all test panels, the cooling system was controlled by a temperature relay that started or stopped the cryostat circulation at a pre-set temperature at the rock-shotcrete interface. Eight temperature sensors were installed in drilled holes in each panel. A temperature sensor was placed to monitor the temperature at the rock-shotcrete interface and the rest of the sensors were located at different depths in the rock and shotcrete as shown in Figure 2. The drill hole used for installation of the temperature sensors was grouted with epoxy. A water access hose was installed in each of the other eight drill holes and these hoses were connected to a bucket filled with water about 1 meter above the test panel (Figure 3).

![Figure 2](image-url)  Simplified sketch of the vertical section of the freeze-thaw lab experiment

<table>
<thead>
<tr>
<th>Panels</th>
<th>Number of Cycles</th>
<th>Number of Days</th>
<th>Temperature at the interface</th>
<th>Access of water</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>20</td>
<td>7</td>
<td>-0.5°C to +0.5°C</td>
<td>Yes</td>
</tr>
<tr>
<td>2</td>
<td>20</td>
<td>20</td>
<td>-2°C to +2°C</td>
<td>No</td>
</tr>
</tbody>
</table>

A reference panel identical to the other panels was prepared. It was not subjected to freeze-thaw cycles. It was used for adhesion tests later for comparison. A summary of the test panels with test conditions is given in Table 1.
2.3 Acoustic Emission Monitoring

Acoustic emission (AE) was continuously monitored during the freeze-thaw tests of the three test panels using a Hyperion Ultrasonic System manufactured by the Engineering Seismology Group (ESG), Canada. The AE system consisted of eight sensors with pre-amplifiers and digitizers (Figure 3). ESG data acquisition system UltrACQ, was used for the real-time monitoring of the AE events. 25 mV threshold voltages was selected for all sensors for the entire test and trigger for recording the AE event was set to 5 sensors. Each waveform was digitized into 4096 samples at a sampling rate of 5 MHz.

The sensors were single-component piezoelectric type, manufactured by Physical Acoustics Corporation (R6α model). The output of the sensors is particle velocity. The sensors have a resonant frequency of ~55 kHz and operate in the frequency range 35-100 kHz and in a wide temperature range -65 °C to 175°C. The sensors were mounted at eight different locations on the top surface of the test panels using silicon high-vacuum grease as a coupling material. The sensors were kept in the position by metal brackets and a piece of insulating material. Prior to AE monitoring of each test panel, UltrACQ was triggered to ensure that the system is working correctly and to check the background noise. A pencil lead test was also carried out to ensure that the sensors were all working properly, the coupling of the sensor to the specimen, and to check the accuracy of the source-location setup and the velocity of the seismic waves.

3 AE events – data and parameter estimation

The total number of the triggered and recorded AE events for Panel 1 and 2 was 1640 and 129, respectively. The minimum magnitude level of the complete recording of the events was estimated from the bending of the Gutenberg-Richter graph to be around -6.
3.1 Data

The AE events were recorded and data was processed and analyzed using the software HSS Suite, a Commercial package designed for industrial seismic monitoring (Engineering Seismology Group (ESG), 2010). This software allows the user to pick the arrival times of P- and S-waves and to calculate the source parameters (location, magnitude, total radiated energy, seismic moment, source radius, etc.) of the AE events. Since this automatic picking did not give satisfactory results all arrival times were then picked manually to increase the accuracy. An example of AE waveforms recorded during the tests with the P- and S-wave arrivals clearly shown is given in Figure 4.

![Figure 4 Examples of waveforms from an AE event with magnitude M = - 5.7 with clear P- and S-arrivals shown](image)

3.2 Source parameters

The source parameters of the AE events calculated in this study are: source location (hypocenter), origin time, magnitude, and energy. The accuracy and reliability of the source localization are influenced by various factors some of which are: the accuracy of the arrival times, the localization algorithm and accuracy of the velocities of seismic waves, and the location of the sensors. The arrival time picking is one of the most important factors. The good signal-to-noise ratio allowed P- and S-arrivals to be picked on almost all channels (see Figure 4). The source location was calculated using both P- and S-wave arrival time data and a simplex algorithm (Nelder and Mead, 1965). This location algorithm assumes a homogeneous and isotropic material, i.e. a constant wave propagation velocity for the whole structure and all directions. For hypocenter localizations the test panels were considered to be homogeneous with an average velocity of the P-waves of 5200 m/s and S-waves of 3000 m/s and Poisson’s ratio of 0.25. The average P-wave velocity was estimated using an artificially generated known source before the experiments started. The S-wave velocity was calculated from the P-wave velocity and assumed average Poisson’s ratio. The accuracy of the hypocenter locations with the average P- and S-wave velocities and the position of the sensors on a plane surface (current position of the sensors) using the data from Hsu-Nielson (pencil lead break) tests (Sause 2011) with known source showed an error of the order of the ± 5 mm.
The results for the source locations of the AE events showed that they were confined in a thin layer around the interface between the rock and the shotcrete. Event locations outside the rock-surface interface were not considered. The estimated mean error for 1593 hypocenter locations for all the experiments was ± 2.4 mm. Even though the sensor locations were on a plane surface with less than 90° azimuthal coverage, the AE locations did not show some systematic shift. Probably the reason for this were the well-defined velocities and the multiple P- and S-arrivals with good quality used for the locations.

The uniaxial magnitude (uMag) was estimated with the ESG WaveVis software. The value was calculated using the unclipped (good) peak amplitudes of a single component trace recorded by each sensor. A correction for the geometric spreading using the source-sensor distance was made using the relation

\[ uMag = A \log(R \cdot ppv) + B \]  

(1)

where ppv is the peak particle velocity in mm/s, R is the distance from the source (hypocentre) to the sensor in mm, A and B are coefficients with default values A = 1.51 and B = 1.24 in the WaveVis software. The magnitude for each event was calculated as the average from all sensor magnitudes. The software does not allow additional correction for the intrinsic attenuation to be applied. Data from 16 Hsu-Nielson pencil lead tests conducted on the test granite blocks were used to estimate the magnitude error due to ignoring this correction. The results showed that using eq. (1) gives larger magnitudes for the sensors which were at larger distances. The estimated difference between the closest and most distant sensors was in the order of 0.2 to 0.3 magnitude units. Hence we were overcorrecting the amplitudes. By averaging the data from sensors at different distances we actually introduced a positive error in the final magnitude in the order of 0.1 to 0.2 magnitude units.

The energy was calculated by the WaveVis ESG software from the integral \( J_c \) separately for P and S-waves using the following formula (Engineering Seismology Group (ESG), 2010; Boatwright and Fletcher 1984)

\[ E_c = \frac{4\pi \rho c R^2 J_c}{F_c^2} \langle F_c^2 \rangle \]  

(2)

where \( \rho \) is the density of the source material in the medium in kg/m³, \( c \) is the seismic wave velocity (for P-wave or S-wave), \( R \) is the hypocentral distance in mm. \( J_c \) is the integral of the squared ground velocity, which by Parseval’s theorem, is the second moment of the power spectrum (Snoke et al., 1987). \( F_c \) is the radiation pattern coefficient (for P-wave or S-wave) which depends on the location of the specific sensor and the orientation of the nodal planes of the focal mechanism of the source. \( \langle F_c^2 \rangle \) is the rms radiation pattern over the whole sphere around the source either for P- or S-wave. In this study the exact value of the \( F_c \) was not known because the focal mechanism was not calculated. Hence the average value for \( F_c^2 \) was used instead. The total seismic energy obtained is the sum of the energy of the P- and S-waves.

In this study, the energy was not calculated directly from the waveforms. Instead, it was calculated from the magnitude (uMag) using a linear empirical relationship with the log of the energy (E) in Joules obtained from the AE data recorded during other experiment on similar granite test blocks, with the same sensors, acquisition system and software (Mainali et al. 2016):

\[ \log(E) = 0.5889 \ uMag - 2.5655 \]  

(3)
with a correlation coefficient of 0.82.

4 AE events - results

The number of the AE events, their locations, magnitudes and energy distributions in time and space were obtained for all three panels.

4.1 Number of AE events

For all 20 cycles of the test in Panel 1, around 90% of AE events were generated during the freeze periods. Around 95% of the total numbers of AE events in this panel were generated during the first 6 freeze-thaw cycles. The interface of Panel 2 had no access to water. For this panel, most of the events occurred at the beginning of the test until cycle 4. In total more than 92% of the events occurred during the freeze periods.

![Epicenters of AE events and drill hole locations for Panel 1. The colour and the symbol of the events correspond to their size (magnitude). The legend for the drill hole type is the same as in Figure 1](image)

4.2 Location of AE events

Figures 5 and 6 show the locations of all AE events with magnitude for Panels 1 and 2 respectively, during 20 freeze-thaw cycles. In Panel 1 (Figure 5) most of the AE events were concentrated in a cluster approximately 100 x 100 mm around one of the drilled holes with geotextile and the rest of the events were scattered over the panel. In Panel 2 the very few AE events were scattered in two large areas.
Figure 6  Epicenters of AE events and drill hole locations for Panel 2. The colour and the symbol of the events correspond to their size (magnitude). The legend for the drill hole type is the same as in Figure 1

4.3 Development of the AE Activity

AE source locations were used to investigate the development of AE activity with time. Figure 7 illustrates the spatial distributions of AE events during cycles 1 to 5, cycles 6 to 10, cycles 11 to 15 and cycles 16 to 20 for Panel 1. In Panel 1, the AE events initially were scattered (cycles 1 to 5) but an area with clustered events started to form. Then the AE events stayed fairly concentrated in the same cluster for the rest of the cycles.
4.4 Energy of AE events

The calculated released seismic energy for individual AE events during freezing and thawing varied from panel to panel and among the cycles. In Panel 1 most of the AE events released energy of up to 1.5E-06 J, both during the freeze and thaw cycles. In Panel 2, the energy was mostly below 7.5E-07 J without substantial difference for freeze and thaw periods.

The cumulative energy of the recorded events for each cycle of each panel was calculated (Figure 8). For Panel 1 the cumulative energy gradually increased from the beginning of the test until the end with some change in the slope of the graph after cycle 14. For Panel 2, the cumulative energy associated with Panel 2 increased slightly until cycle 3 and then stayed almost constant. The final accumulated energy in Panel 1 was approximately 27 times larger than in Panel 2.
5 Adhesion test - results

Once the test panels had been exposed to the freeze-thaw tests, eight cores, concentric to the holes for water supply (5 mm diameter) and with a diameter of 94 mm, were drilled from each panel. These rocks – shotcrete interfaces were subjected to tensile loading and the adhesion was evaluated. The adhesive strength of the tested interfaces showed a large scatter and did not show any strong correlation with surface treatment, water conditions during the test or freezing-thawing characteristics. The same scatter is observed for the shotcrete-rock interfaces tested by Saiang et al. (2005).

6 Discussion

The aim of the AE monitoring in this study was to identify the intensity and the spread of the deterioration that occurs in the rock–shotcrete interface due to ice pressure. In this case, the AE signals were used as an indicator of the actual crack growth/debonding using their number, location, magnitude and the seismic energy.

In Panel 1 the AE events continued through the whole test and were concentrated within a small cluster (Figure 5) around one of the drill holes prepared with geotextile. This implied possible cracking and/or debonding within a limited area. The expansion of the area with AE events (Figure 5) indicated that the cracking and/or debonding area slightly increased during the test.

The AE activity in Panel 2 was considerably lower than that in the other test panels. Most of the events occurred at the beginning of the test until cycle 4. The energy and the magnitude of the AE events were also low compared to those registered in the other panels.

The energy (or magnitude) range of the AE activities depends on the nature and source of cracking/debonding. The larger energy and magnitudes associated with most of the events recorded in Panels 1 is probably a result of the debonding of the shotcrete-rock material while the smaller events recorded in Panel 2 are micro-cracking developed at the interface or in pores due to ice expansion or induced by the ice formation itself. There is one other possible source of the AE
emission – the ice forming or thawing itself. Previous studies showed that AE can be recorded during crystallization and melting (e.g. Sakharov 1994; see also the references there).

7 Conclusion
The results showed that:

- Most of the acoustic emissions occurred during the freeze cycles.
- More AE events with higher energy and magnitudes occurred in the panel with access to water during freezing-thawing (Panels 1) compared to those without access to water (Panel 2).
- In general, the number of AE events and their magnitude decreased with increasing the number of freeze-thaw cycles.
- The spatial distribution of the AE in the later cycles took place in the areas with previously observed AE during earlier cycles of freezing and thawing.
- The transition of water into ice and the associated expansion causes deterioration in the rock-shotcrete interface. The deterioration depends on the freezing rate, freezing intensity (temperature range), degree of water saturation, and the duration of the water flow. This makes it difficult to interpret which factor is most critical for the damage in the rock-shotcrete interface.
- Most probably the observed AE are caused by three different processes: debonding, micro-cracking and ice forming/melting.
- The adhesive strength evaluated from the test and reference panels showed a considerable scatter (Andrén, 2009) which has also been shown for “virgin” samples in an earlier study (Saiang et al. 2005).

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References


Local seismic systems for study of the effect of seismic waves on rock mass and ground support in Swedish underground mines (Zinkgruvan, Garpenberg, Kiruna)

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Abstract

Three local seismic systems were installed by August 2015 in deep underground mines in Sweden – Zinkgruvan Mine (Lundin Mining AB), Garpenberg Mine (Boliden AB), and Kiirunavaara Mine (LKAB) as part of a project for developing new methods for Evaluating the Rock Support Performance (ERSP, Vinnova). The areas were chosen within the most probable volumes where large rockbursts can be expected. The local systems were installed at mine levels between 730 and 1150 m in different mines. The horizontal extend of each instrumented areas is between 70 and 100 m. The seismic system in each mine is a combination of uniaxial and three-axial 4.5 Hz geophones installed on the surface, in shallow (~0.5 m) and deeper (6-9 m) boreholes in profiles across drifts. These profiles are in close proximity to profiles with extensometers, instrumented bolts, and observation holes. The seismic systems are manufactured and installed by the Institute of Mine Seismology (IMS). The aim of the seismic systems is to record the seismic events that occur in the vicinity of the instrumented areas and provide valuable data about the variability of seismic waveforms around the underground openings and changes when seismic waves approach them. Data is used to study: 1) the attenuation/decrease of the maximum ground velocity (PPV) with the distance, especially at small distances; 2) site effects, including maximum amplitudes, predominant frequency, and duration of the seismic signals, 3) the attenuation/amplification of the seismic waves approaching the underground opening. The final aim is to obtain new information that can be used for improved requirements for the rock support design in rockburst prone areas.

The installation of the seismic systems started in May 2015 (Zinkgruvan Mine) and was completed by August 2015. They run mostly in triggered mode with initial automatic arrival time picking and source parameter calculation and subsequent manual processing of seismic event of interest. More than 200,000 seismic events with magnitude from -4.5 to 2.0 were recorded by December 2015. At present only a small portion of all data was processed manually and the procedures for processing of the events were developed on this subset. The first results from the monitoring showed that there are differences in the amplitudes and shape of the seismic signals recorded by the sensors installed in deeper borehole (behind the most blast-damaged zone (6 – 9 m)) and close to the surface (0.5 m) or on the surface of the openings. There are also differences between the waveforms recorded on the walls and the roof along the same profiles or on nearby profiles. Data from the investigated rockbursts showed maximum velocity recorded from a seismic events at close distances with magnitude larger than 0.5 in the order of 10 cm/s with clipping levels 10 – 20 cm/s.

1 Introduction

Rockbursts are major safety issue in deep underground mines around the world. They are also a problem for deep underground mines in Sweden. To reduce the hazard due to rockbursting one of the most important factors is the adequate rock support. The optimal rock support should be designed according to the specific conditions in the mines and real ground motions generated by the induced seismic events in
the specific mine. A project ESRP was initiated in September 2014 by the Lulea University of Technology and three Swedish mining companies (Lundin Mining, LKAB and Boliden AB) in order to collect new data about the ground motion from larger seismic events together with data from different types of geotechnical instrumentation (Vinnova, 2013). The main aim of the project is developing new methods for Evaluating the Rock Support Performance (ERSP) that use all available seismic and geotechnical information in each mine. Mine-specific relationships between the source parameters of the seismic events, local variations of the ground motion produced by the seismic events, and the factors affecting the damage to the underground openings and the rock support are expected to be developed as a result of the project (Zhang et al., 2016).

The first results presented in this paper are based on the data obtained from three local seismic systems installed as part of the ESRP project, combined with data from the existing permanent seismic systems in the participating mines. They include: 1) attenuation/decrease of maximum velocity of the ground motion (PPV) with the distance; 2) local variations of the ground motion caused by local site effects (differences in the maximum amplitude, duration, and predominant frequency); 3) attenuation/amplification of the seismic waves approaching the underground opening.

2 Local seismic monitoring systems

Local seismic systems, consisting of 16-18 sensors were installed in three underground mines in Sweden – Zinkgruvan Mine (Lundin Mining AB), Garpenberg Mine (Boliden AB), and Kiirunavaara Mine (LKAB) in May to August 2015. The sites were chosen within the volumes where larger seismic events and rockbursts were expected to occur based on previous seismic events or expected production development. The mine level of the areas where the local systems were installed varied between ~1150 m (Zinkgruvan), 1100 m (Kiirunavaara), and 730 m (Garpenberg) (Table 1). Geophones with natural frequency 4.5 Hz were installed within comparatively small horizontal span (70 to 100 m) on vertical profiles crossing the underground openings. The local seismic system for Zinkgruvan and Garpenberg Mine are shown on Fig. 1. The layout of the seismic system in Kiirunavaara (Kiruna) mine is presented in Zhang et al. (2016).

### Table 1 Summary of the seismic local systems as of 31 December 2015

<table>
<thead>
<tr>
<th>Mine name</th>
<th>Zinkgruvan</th>
<th>Kiirunavaara</th>
<th>Garpenberg</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Basic information</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Owner</td>
<td>Lundin Mining</td>
<td>LKAB</td>
<td>Boliden AB</td>
</tr>
<tr>
<td>Orebody at experimental site</td>
<td>Burkland</td>
<td>Kiirunavaara</td>
<td>Lappberget</td>
</tr>
<tr>
<td>Depth at experimental site</td>
<td>1150 m</td>
<td>1108 m</td>
<td>728 m</td>
</tr>
<tr>
<td>Local seismic monitoring system*</td>
<td>12U+6T</td>
<td>12U+4T</td>
<td>12U+4T</td>
</tr>
</tbody>
</table>

*U indicates uni-axial geophones and T indicates tri-axial geophones.

Most of the sensors are uniaxial. They were installed on the surface of the openings (Zinkgruvan) or in shallow holes (50-60 cm from the opening behind the most damaged rock mass) (Garpenberg, Kiruna), with their axes oriented perpendicular to the opening. There are also sets of triaxial sensors installed in pairs – one in deeper boreholes (6 to 9 m) and the other in shallow boreholes (50-60 cm) (Fig. 1). The aim of the surface/shallow sensors was to provide input for analyses of the wave field close to the excavations and to study the local variations and site effects of the seismic waves. The aim of the deep/shallow pairs of sensors was to study the transfer function (modification of the seismic waves when they approach the opening/free surface). Data from all sensors was used also to study the attenuation/decrease of the seismic waves (peak particle velocity – PPV) with the distance, combined with the data from the seismic sensors of the existing permanent seismic systems in the mines and to establish relationships between the seismic source parameters and the rock mass and rock support damage due to rockbursts.
Figure 1  Schematic diagram of seismic local system in Garpenberg Mine (top) and Zinkgruvan (bottom).
The signal from the seismic sensors is digitized with frequency of 6 kHz (Kiirunavaara Mine) or 12 kHz (Zinkgruvan and Garpenberg Mines). Data is collected and transferred by IMS (Institute of Mine seismology) equipment to the server on the surface. Timing of the seismic system is either synchronized with the timing of the permanent seismic systems (Garpenberg and Kiirunavaara Mines) or it is Internet synchronized (Zinkgruvan Mine). The systems run mostly in triggered mode, with remote access to the data. The software for data processing is provided by IMS.

After the installation and recording for a few months was found that the clipping level of the geophones (~10-20 cm/s) was reached for some seismic events. The gain of a few sensors at each site will be decreased in order to increase the clipping level and to be able to record larger ground motion velocities.

The geomechanical instruments for the project, which include Multiple Point Borehole extensometer (MPBX) and Instrumented bolts (from YieldPoint, Canada) were installed in proximity (distance up to ~50 cm) to the seismic systems. Observation holes for 360° digital optical scanning by Slim Borehole Scanner (SBS) (DMT) were also drilled. Laser scanning technique is used to monitor the closure of the excavation. For a detailed description of all instrumentation see Zhang et al. (2016).

3 Seismic data and processing

During the period from May/August 2015 until December 2015 each one of the seismic systems recorded large amount of very small to larger seismic events (magnitudes from −5.0 to 2.2). The seismic systems run mostly in triggered mode with initial automatic arrival time picking and source parameter calculation. The manual verification of the results showed that the accuracy of the automatic arrival time picking was not enough for precise localization of the hypocenters and it was decided to process manually a subset of events. The criteria for choosing events were the number of sensors that recorded the event (> 15) and if any sensor recorded an event with PPV (Peak Particle Velocity) > 5 mm/s. (The clipping level was ~10-20 cm/s.)

To get reliable seismic source parameters manually picked arrival times from the local seismic systems were combined (merged) with the data from the existing permanent seismic systems and processed together. IMS Trace software was used for the initial routine processing and calculation of source location and dynamic source parameters. As the number of sensors and the geometry of the permanent systems are very different in each mine, and time synchronization was available only in Kiruna and Garpenberg mines, there were some modifications in the processing approach for each mine. Merging of the data was straightforward for Kiruna Mine, which has the largest number of permanent sensors (~230) and the best sensor coverage. The hypocenter locations and estimated size of the events in this case did not differ substantially between the permanent seismic system and the merged data. For Garpenberg Mine the comparison of the results for the hypocenter locations from the permanent system and the merged data showed very large differences depending on the azimuthal coverage (Erguncu Güçlü, 2016). In some cases the differences in the hypocenter locations between the two systems was in the order of 200 m with the large difference in the hypocentral depth. In this case special investigation was carried out to find the best solution. The results from the merged dataset were chosen as final for Kiruna and Garpenberg mines. For Zinkgruvan Mine only data from the local system was used for calculation of all source parameters as the permanent seismic system is very sparse and the closest seismic sensors are far from the local sensors and there is no time synchronization between the local and permanent seismic systems.

The first results presented here are based on data from events recorded in the three participating mine until the of December 2015.

4 PPV decrease with the distance

The peak particle velocity (PPV) is an important parameter which is used in the underground support. The PPV values obtained by the permanent seismic systems in the three participating mines in the best case scenario are at hypocentral distance around 100 m. Data for the near field, close to the seismic sources are
very sparse. The installed local seismic system provided unique opportunity to obtain data very close to the hypocenters of the seismic sources, in some cases at 10 m, and to fill in the gap in the near field. The largest PPV that were recorded on unclipped records were in the order of <10 cm/s for seismic events with magnitude > 0.5 and distances up to 100 m. For the seismic events with magnitude between 0.0 and 0.5 the PPV was up to 25 mm/s. (The magnitude used in this study is the local IMS magnitude, calculated from the seismic energy and potency.)

The decrease in the PPV was studied by IMS for Kiruna Mine (de Tout et al., 2012) and two empirical relationships were derived from the data from the permanent seismic system. These relationships were tested on the data from both, permanent and local seismic systems for all three mines (Fig. 2).

The results showed that relationship derived for the potency as a parameter in overall fits better the data from all mines. The empirical relationships using energy as a parameter are underestimating the real data. Even though the relationships were derived for data from Kiruna Mine they fit quite well the data from Garpenberg and to some extent from Zinkgruvan. As only data from the local seismic system in Zinkgruvan were used, the conclusion is based only on data at close distances. These first results give indication that the PPV decrease at short distances up to 100 m is very fast compared to the decrease at larger distances. This means that data at larger distances cannot be extrapolated to obtain the PPV decrease at short distance. It has to be noted that the empirical relationship in de Tout et al. (2012) is extended at short distances using theoretical considerations not real data.

The work will continue with more data and with a goal to find empirical relationship for the PPV for each one of the mines as a function of the potency/seismic moment, magnitude, and seismic energy.
5  Local variations of seismic spectra

One of the aims of the project is to define the variability of the seismic waveforms and their parameters as amplitudes, frequency content, and resonant effects recorded in close distances around the underground opening, e.g. possible site effects. The initial stage of the analysis included only visual inspection for substantial differences in the frequency content and possible resonances. We looked in which cases (hypocentral distances, magnitudes, and azimuth of the rays from the source to the sensors) there are effects that are noticeable visually.

Examples of stacked S-wave spectra for seismic events recorded in Zinkgruvan and Kiruna Mines are shown on Fig. 3.
b)

Figure 3 Stacked S-wave spectra for two seismic events recorded at (a) Zinkgruvan mine: $M = -1.9$ (distance 144 to 158 m) (left) and $M = -3.4$ (distance 34 to 42 m) (right), and (b) Kiruna Mine: $M = -1.8$ (distance 8 to 68 m) (left) and $M = -1.8$ (distance 61 to 118 m) (right). The inserts show the azimuthal distribution of the raypaths.

The results from the three datasets from different mines showed that the spectra in Zinkgruvan Mine have more variability than in the other two mines. In some cases the differences in the spectral level is 7-9 times. The sensors with larger amplitudes are not always the same and the frequency at which larger amplitudes are observed varied. There is also indication that even though the differences in the spectral shape and level in the other two mines (Kiruna and Garpenberg) are not so large, there are still visible and more variability is observed when the azimuthal paths differ more (Fig. 3b, left) than when they are closer (Fig. 3b, right). In Zinkgruvan Mine the variability in the spectral shape is observed even in cases of comparatively close raypaths. The most plausible explanation for the differences in the behaviour of the spectra in Zinkruvan Mine compared to Garpenberg and Kiruna Mines, could be that the sensors in the first one are installed directly on the surface of the opening after removing the shotcrete while the sensors in the other two mines are installed in shallow holes (50-60 cm) from the opening. More sensors will be installed in different conditions close to each other to verify this plausible conclusion.

The work will continue with statistics on the differences of the spectral level and dominant frequencies as a function of the magnitude of the seismic events, the distance and the azimuth. The difference in the waveforms recorded in the time domain will be studied also. An example of different records from an event recorded in Zinkgruvan Mine at almost the same distances is shown on Fig. 4.
Figure 4  Example of waveforms from seismic event with $M = -3.4$ recorded in Zlnkgruvan Mine at distances from 34 to 43 m.

6 Transfer function

At every site/mine there are at least two sets of triaxial sensors installed, each set consisting of one shallow sensor and one in a deeper borehole 6 to 9 m from the opening. These pairs of sensors are used for study of the effect of the free surface/opening on the seismic wave characteristics (amplification, reflection, etc.). To obtain the transfer function the S-wave spectra on different components were calculated and the ratio of the spectra between the surface/shallow and deep sensor were calculated.

Examples of the original records and the spectral ratios for each component (one vertical and two horizontal) are shown on Fig. 5.
Figure 5  The original records from both surface and deep sensors (left) and the S-wave spectral ratios component by component (right) (X and Y are horizontal components and Z is vertical component). The pairs of sensors are on the roof of the opening (top) and on the wall on the same vertical cross section. Data is from an event with M = -0.2 recorded at distance 78 - 80 m in Kiruna Mine. The amplitude scale is different for different records and is shown in the left upper corner of each record (MaxAmp).

The first results obtained for the spectral ratios are very consistent. They show almost no difference in the spectral level of the records from the shallow and deep sensors for up to 100 Hz. Between 100 and ~1000 Hz the spectral level of the records from the surface sensors is up to 10 times larger than that of the deeper sensors and above 1000 Hz this level is a few times smaller. The further work will try to find a physical...
explanation of the observed phenomenon. More work will be done on the statistical representation of the results obtained from larger number of records and in different conditions in different mines.

7 Conclusions

The first results obtained from the local seismic systems installed in three different Swedish mines – Zinkgruvan, Garpenberg, and Kiruna showed differences in the seismic events recorded at close distances up to 100-200 m in a few different aspects: variation of the waveforms recorded almost at the same distances but in different positions (roof or wall) and at different azimuths from the seismic sources. The corresponding spectra also differed but mostly for the sensors in Zinkgruvan Mine, which were installed on the surface of the opening, and less in Kiruna and Garpenberg Mines with shallow sensors installed at 50-60 cm deep holes. An interesting result obtained so far is the amplification/de-amplification of the seismic waves coming towards the opening, different for three frequency bands: up to 100 Hz – no amplification, 100 to ~1000 Hz amplification of almost 10 times, and above 1000 Hz de-amplification a few times. The results for the PPV decrease show that the new data fill in the gap for very small hypocentral distances and can give valuable information about the changes in this parameter for these distances.

The work will continue with more processed data and statistical representations of different kind of results to find the dependence on the magnitude of the seismic events, possibly the mechanism of the seismic source, the distance and the azimuth of the raypaths. It has to be mentioned that in many cases (for larger seismic events) the data recorded by the local systems are within the source radius (so-called ‘near field’). In this case the usual representation of the seismic source as a point source is not acceptable and the work has to be done with more complex representation of the seismic source.

The results of this study and within the ESRP project can give valuable empirical data for improvement of the understanding of the seismic wave field near the seismic sources in the mines and subsequent improvement of the principles of the rock support design in the mines.

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Methodology for incorporating new elements of support for the mine design in El Teniente mine

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Abstract

El Teniente Mine has over 100 years of history in mining, during this period the rock support systems have had to adapt to environmental conditions, migrating from a passive reinforcement based on wooden frames in a secondary rock, to an active reinforcement using steel bars and cables, this in response to the increased stresses as a consequence in the change of rock features and the deepening of mining activities.

This paper aims to briefly describe the methodology and process of inclusion of new elements of fortification that allows them to be part of the mine design is adding to the support of tunnels and how the concepts of energy dissipation have been determining the use, characterization and interaction of elements and ground support systems in recent years.

In the 70s it was common to see wood-based reinforcement, related directly to the extraction of secondary rock mass and the necessity of a reinforcement that could support static disarmament of poor quality rocks. In the early 80’s, during the operation of Teniente 4 Sur mine in a primary rock mass (hard rock), started the systematic use of steel bars, along with complementary elements such as plates and nuts, and by the end of the decade electro-welded meshes and shotcrete is added to the support systems, responding to the need of retention between the reinforcing elements. Afterwards and by the hand of dynamic loads, at the beginning of the 90s, it is sought to increase the capacity of dissipate the energy of the retainers elements, migrating to woven meshes that allowed greater deformation. Moving to the next decade the analysis of systems have improved and progress has been made in testing elements, classifying reinforcement elements according to their ability to dissipate energy. As a result, reinforcement systems have been designed capable of withstanding greater loads both static and dynamic, used in areas of greater complexity in the mine.

As described in the previous paragraphs, in relation to changes or evolution of support system, it is necessary to add new elements, because the deepening, rock types, methods of operation, etc., create new needs and for this to be done properly, a methodology to correctly handle the inclusion of new elements is required.

1 Introduction

In El Teniente Mine, geomechanical problems of various character are recognized, however, the more complex phenomenon and which generates a greater impact in the mining operation is the rock burst, mainly for its damage consequence to both the mine infrastructure and the people. These damages also affect, undoubtedly, to the fulfillment of the production goals, which detriment the mining business.

Since the eighties to date, with the aim of tackling the problem, it has been implemented a series of mitigation measures, which have addressed both from a strategic point of view, and as tactician. In strategic terms, measures have been implemented associated with the
development and improvement of the mining methods, the mining rate control, the installation of seismic monitoring systems, and the incorporation of hydraulic fracturing, among others. Regarding the tactical order measures, these have been associated with the change in the mining designs and improvement of ground support systems drifts, which basically this report is about.

Further to the characterization of the elements, that will be described in this report, it has been constantly considered the evaluation of the ground support installed in the productive sectors, which has been continuously in stages of improvements and updates based on the requirements evidenced in each sector, which together with other mitigation measures have allowed to increase or maintain the production rates and reduce the occurrence of rock bursts at El Teniente mine.

The following chart shows the number of rock bursts versus the primary mineral mining, where it can be seen the decrease of the frequency, due to mitigation measures which goes hand in hand with the understanding of the problem.

![Figure 1 Graphic number of rock bursts versus primary mineral mining](image)

The importance of the reinforcement is directly in being the last contention barrier between the released energy and the infrastructure and/or the personnel operating in the mine, and it is for this reason that the efforts to strengthen and keep the damage associated with rock burst are of great importance.

As part of the evolution of the ground support systems in El Teniente mine, that actually constitutes understanding the concept of system, however, the first step in this development is the individual knowledge of each element that makes up this system.

Afterwards, thinking of keeping the catalog updated with information of the reinforcement elements, it was generated an evaluation methodology for the entry of new reinforcement elements to the catalog, which involved: first, finding the elements in the market, then it is contemplated conducting a series of tests to the elements, tests type: static, dynamic, corrosion testing, and operational testing ground, among others, in order to record useful data for setting the ground support systems. Finally their behavior is evaluated in real field conditions inside the mine.

This catalog, developed from a series of tests, provides us with the necessary information to generate support designs suitable to the needs of different sections of the mine.
All this information corresponds to the basic input for the design of ground support systems, according to the energy, deformation and corrosion requirements, estimated necessary by the designer.

So it is described in this report, the results of the catalog of elements and how that information has helped to control rock burst damage, in the same way it is presented a study case that estimates the capacity from the elements of the catalog.

1.1. **El Teniente Mine overview**

El Teniente Mine is the world largest copper-molybdenum underground mine owned by Codelco-Chile. It is located in the Andes range in the central zone of Chile, about 70 km South-Southeast from capital city, Santiago. The current production is 140,000 tonnes/day. The mine is in operation since 1905. Since the 1940’s, exploitation has been conducted applying caving methods. Up to the middle of the 1970’s, the rock mass under production was a low mechanical competent rock mass, the secondary rock mass. Since then, a more competent rock mass has been exploited, generating a relevant seismic response. Basically more than 100 years of a long history of mining, where ground support systems also had to adapt to each of the requests that the rock mass and exploitation imposed.

![Figure 2 Location of the El Teniente mine, Codelco-Chile.](image)

2 **Conceptual framework**

In recent years the systematically reduce of rock bursts occurrence has been managed, deeper and without reducing mining rates. An important factor in these results has been the continuous improvements and studies that have been developed in terms of ground support, understanding that in front of static or dynamic loads, the ground support systems are the last barriers that allow keeping the shapes of the drifts, protecting the integrity of the people and allowing the operational continuity.

After each rock burst occurred, a number of "retrospective analysis" are made, which seeks to determine relevant information to understand the failure mechanisms involved in the generation of the damage. One of the parameters that has been searched to estimate, is the energy required to generate the damage in the evaluation sector. Depending on the local conditions of the areas damaged by a rock burst, the number of assumptions that must be performed for these estimates is evaluated, resulting quite reliable data in several occasions.

The knowledge of the energy demands by means of this or other techniques, allows estimating the energy that must be dissipated by the reinforcement system to install. This motivates a
complete characterization of the elements of ground support used in El Teniente mine and of several reinforcing and retention elements available on the market, in which all features are included. Particularly the features defined as relevant by the designer: for example its ability to dissipate energy, deformation ranges and corrosion resistance.

In El Teniente mine has begun the installation of ground support systems relating with the energy demand versus the capacity to dissipate energy, in addition to limiting the maximum distortion at the edges of the excavations, as well as considerations of durability over the time in corrosive environments since drifts in a panel caving mining exceed a decade. Special care exists in the interaction among the elements of the reinforcement (plate, nut and bar) and their interaction with retaining elements as meshes, shotcrete and “corchetes” (camps).

3 Catalog and methodology

A collection of multiple elements or devices used in the mine was generated to the stabilization work of underground excavations and the relevant information for each product was ordered according to its main characteristics and the role they play in a global system of support. Once fully characterized the elements used in the mine, the following selection and assessment methodology was developed to complement the data catalog required for the design of reinforcement according to the criteria adopted. On the lines below the methodology developed is summarized:

![Figure 3  Scheme of the methodology for selection of support elements in El Teniente](image)

3.1 Stage 1: Supplier Information

The first stage of this methodology corresponds to the action of compiling the information provided by the supplier, where features of the element are explicated, which are important to highlight from the supplier’s product. In this way it is intended to have a first idea of the nominal performance of the elements and the variability in the functional characteristics of the purchased items. It is noteworthy that the support elements in evaluation have information corresponding to its static and dynamic capacity in their respective catalogs, nevertheless, there are not many that specify the parameters used in the tests by the responsible laboratories.

3.2 Stage 2: Testing to elements/systems (laboratory).

In this second stage, and basically due to the uncertainty that has the uniformity of information associated with the test of each element, it is considered appropriate testing under conditions equivalent to each element. To this stage there were basically defined based
tests for static and dynamic capacity of the element and its resistance to corrosion, both national and international laboratories, in order to unify the evaluation parameters. It is important to say in the case of the bolt, for example, that although the anchoring element in rock is represented by the bolt itself and the bounding element, each of these components have associated its nut and plate, which are also evaluated, so to set up a complete reading of the system behavior [bolt/plate/nut]. In field tests the main challenge is to determine the adhesion between the element and the bounding (grouting), a good example are the pull tests to bars and cables.

3.3 Stage 3: Operational Assessment

In addition to the constituent and functional characteristics of the elements, it is important to assess the ability of installation inside the mine, in order to generate the least impact in mining operations. Therefore, it is considered within the pattern evaluation of this methodology, the operational component of the support system. In this item, the results are obtained by developing the field installation of the elements, with the purpose of capturing relevant information in operational terms, such as required drilling diameter, installation cycle times, associated risks, technical requirements, etc. Specifically for this operational test of the elements it has been generated partial installation tests of all purchased elements. Subsequent to this stage, in which different results are evaluated, it is estimated to have enough information to recommend one or several elements.

3.4 Stage 4: Post evaluation of the element

The last stage of this methodology, refers to in the context of industrial installation of one or several selected items. The basic criterion to define the introduction of a support element is that its skills evaluated in laboratory are higher than those of the reinforcement that is currently used in El Teniente mine. Regarding this last stage, the purpose is to capture direct information from the field, associated to the operation of the support system in conditions of habitual operation in the mine, with a focus on installation of one or more support elements in vulnerable areas, in order to verify the operation of the element under real conditions. This step is strongly supported by specific monitoring systems.

This newly exposed methodology was used extensively to evaluate a series of bolts available on the market, which were acquired as anchoring elements of “yielding type”, the results of the analysis are mentioned below.

3.5 Case base, helicoidal bolt diameter 22mm, A440-280H

The helicoidal bolt A440 280H, of 22 mm in diameter, is the basis for our selection criteria. That is, every element to be incorporated, should have a superior performance to the base.

The bolt used extensively today in El Teniente actually corresponds to the helicoidal bolt diameter 22 mm, steel A440-280H. These are cross section bolts, slightly oval, with projections in the form of a left helicoidal thread, these are supplied in straight bars, in the state of hot rolling and without further treatment.

It is used as a reinforcement and support in the ground support system of almost 100% of the drifts which are developed and that have developed in the mine in the last years.

The helicoidal bolt or helicoidal bar works together with plate and nut, the system is clearly specified in internal planes of the mine. One or several bolts are installed as part of the support system, in order to improve the dynamic behavior of this system, must at least overcome the static and dynamic performance to the helicoidal bolt.
In addition to this methodology evaluating corrosion of elements is added, since the bolts are installed in highly corrosive environments, it is important to have an assessment of this type.

### 3.5.1 PROPERTIES

**Table 1 Mechanical properties steel bar (bolt)**

<table>
<thead>
<tr>
<th>Characteristics</th>
<th>Capacity</th>
</tr>
</thead>
<tbody>
<tr>
<td>Minimum for the yield stress</td>
<td>280 [MPa]</td>
</tr>
<tr>
<td>Minimum for the breaking stress</td>
<td>440 [MPa]</td>
</tr>
<tr>
<td>Breaking lengthening</td>
<td>16%</td>
</tr>
</tbody>
</table>

**Table 2 Capacity standard bar 22[mm]**

<table>
<thead>
<tr>
<th>Characteristics</th>
<th>Capacity</th>
</tr>
</thead>
<tbody>
<tr>
<td>Yielding load</td>
<td>98 [kN]</td>
</tr>
<tr>
<td>Breaking load</td>
<td>157 [kN]</td>
</tr>
<tr>
<td>Breaking lengthening</td>
<td></td>
</tr>
</tbody>
</table>

**Table 3 Dynamic Capacity, evaluated at WASM**

<table>
<thead>
<tr>
<th>Characteristics</th>
<th>Capacity</th>
</tr>
</thead>
<tbody>
<tr>
<td>Energy Dissipation</td>
<td>22 [kJ]</td>
</tr>
</tbody>
</table>

The bolts are considered an important link in the system, because through these it is developed the strain of the main reinforcement from the anchor in the stable rock area.

Secondary devices or area elements covering the free faces of the excavation are able to develop load as they react with the bolts. The other devices of the overall system also are attached to the bolts so that the volume of rock mass displaced by a potential dynamic event, for example, is distributed by these devices in its entirety to the bolts anchored to the rock. In conclusion, the bolts serve two essential tasks for the proper functioning of the overall ground support system: strengthen and support.

### 4 Results

In summary, both catalog reinforcement elements used in El Teniente, and the methodology for the selection of new elements, aim to determine under one same criterion their functional abilities, so to have the data and enough operational information to implement and evaluate ground support systems, in order to generate, for example, different configurations that allow satisfying energetic and static demands in a given sector.

To clarify and summarize a part this information of interest, the following charts were generated:
Figure 4  Graphic of static capacity of the tested bolts

Figure 4 shows a preview of the static results of the reinforcement bolts, particularly the load failure of each element is schematized, which have been tested under the same boundary conditions, assessing the capacity of the bar, together with its plates and nuts.

Figure 5 shows a preview of the dynamic results of the fortification bolts, particularly the energy dissipated is indicated and the displacement registered in tests made in Australian laboratories, which have been tested under the same boundary conditions, assessing the capacity of the bar, together with its plates and nuts. The area of desired elements is shown in gray, limiting maximum deformation.

Figure 5  Graphic of dynamic capacity of the tested bolts
Figure 6 Graphic of corrosion rate of the tested bolts

Figure 6 shows the corrosion rate experienced by the different types of bars, in relation to the helicoidal bolt, i.e. it corresponds to normalized values.

5 Design

Energy balance,

\[
\text{Energy} = \text{installed capacity} = F.\ S. \times \text{Energy demand}
\]

To estimate the available energy in a drift, the individual contribution of each element is assumed, assuming ideal conditions regarding to the load mode (tension) and thus the way in which the energy will be dissipated. Meanwhile, the energy demand is estimated by the designer.

It is necessary to limit the maximum displacements of reinforcement and retention systems, this is taking into account the operational continuity that the productive sectors require, the size of the equipment in relation to the size of the drifts and the historical evidences after the reinforcement loads.

For the design of the reinforcement, the systems should use elements that have been evaluated by the protocol described and the following acceptable ranges are considered for their use in the El Teniente mine.

- Minimum energy able to dissipate by the elements of reinforcement should not be less than 22 kJ for bolt of systematic use in the reinforcement and 8 kJ/m² for meshes.
- Maximum displacement of the elements of the reinforcement should be limited to 200 mm in reinforcing elements and 300 mm/m² in meshes.
- Minimum durability in acidic environments such as El Teniente mine, must demonstrate durability equal to or greater than the elements currently used (Helioidal, Teniente bolt).
The different combinations or configurations of the reinforcement systems (for example, pattern variations) or interactions with different retainer elements, are resulting in different energy dissipation capacities, which must be balanced by the designer according to the estimated demand for that sector.

These criteria are complementary to static balance, traditionally used for the estimation of the ground support.

6 Study case

On March 28, 2015, during the course of shift B (approx. 18:30hrs), the occurrence of a seismic event of magnitude 1.8 Mw is recorded, affecting the drift of the South Access of production level of Reservas Norte-Corbata mine.

The focus of the seismic event was registered in an area of geotechnical complexity, characterized by the presence of G Fault system and East-West Fault System, in addition to Dacita and CMET lithologies in their contacts.

![Figure 7 Location in Reservas Norte-Corbata mine of the described seismic event](image)

Being an access route and for the environmental conditions in relation to the proximity to the cavity, stresses conditions and geology of the area, the drift had definite reinforcement, which is complementary to the development one (bolts, mesh and shotcrete), such ground support included a second mesh across the drift, “corchete” cables in walls and debonded cables in the roof and shoulders, giving a capacity of available energy to dissipate above 45 km/m² (energy from the catalogs).
The focus of the event was located a few meters from the drift and it registered a magnitude and energy high enough to cause a rock burst, in relation to the historical occurrence in the mine. The ground support was evidently loaded, rock displacements and local reinforcement element failures, however the entire ground support system was able to keep and mitigate the consequences of the event.

7 Conclusions

It is important and necessary to have a complete characterization of the elements and/or reinforcement systems depending on the particular conditions of use.

Regarding the evolution of the ground support in El Teniente, the reinforcement design changed from static criterion reinforcement to a dynamic one.
Another important development, was that initially the design of the reinforcement considered the elements individually, focusing on the reinforcement and regardless the retention. Currently, in addition to verifying the interactions and the ability to transmit strains between the elements, the ground support is considered as a system, it means, there must be adequate interaction between retaining and reinforcement elements. For example: rebar, nut and plate system interacting with meshes and shotcrete.

The methodology described in this document regulates the inclusion of new elements facilitating decision making that impact in the long term.

Adequate and complete characterization of the elements, it facilitates and supports the new designs, providing better tools to the designer.

8 References

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Methodology for stability analysis of entry-type underground excavations

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A. Delonca, Advanced Mining Technology Center (AMTC)- University of Chile, Chile

Abstract

The potential for instability in the rock surrounding underground excavation is an ever-present threat to both the safety of workers and equipment. In order to counteract these threats, the design of support in underground excavation can be used. However, it presents complex problems of stability, given the uncertainty associated with the engineering parameters and the behavior of the rock mass. This paper proposes a methodology for the stability analysis of typical entry-type underground excavation in pre-feasibility stage. It consists in numerical modeling considering different geomechanical parameters of the rock mass and different support designs. Several numerical models have been defined based on the depth of the excavation, the ratio of the horizontal and vertical stresses K, the rock quality and the uniaxial compressive strength. Moreover, for each case modeled, a specific ground support system is considered consisting in a bolt and/or layers of shotcrete. The design of the supports has been estimated using an empirical support guideline. The state of stability of each model has been evaluated considering the Shear Strength Reduction (SSR) method and a proposed stability criterion. Until now, the SSR method was used as an analysis technique in mining slope stability for open pits. In this study, it was coupled to a stability criterion in order to propose a valuable tool to the evaluation of the underground excavation stability in pre-feasibility stage.

1 Introduction

To date, the security and protection of both the workers and equipment are the most important challenges facing the underground mining industry. It is thus needed to develop methods to improve design and stability of underground excavations.

The numerical modeling is a useful tool that allows the engineering problems to be better represented. It is based on mathematical expressions, which simulate the behavior of an object of study. It can be used to describe the rock mass behavior and thus to study the stability of an underground excavation. The shear strength reduction method, SSR, is a method mainly used to determine a safety factor through the progressive reduction of strength parameters of the material, bringing the model to a state of equilibrium.

This paper proposes to develop a new methodology to evaluate the stability of typical entry-type underground excavation in pre-feasibility stage when reinforced with a support system. It is based on the use of the SSR method into numerical modeling. Different scenarios of analysis are defined, based on the rock mass characteristics and the rock support. Then, the SSR method, as well as the numerical modeling, are used to propose a stability criterion in the different scenarios. The adjustment of this method will lead to evaluate the stability of the excavation based on the support considered (rock bolts and shotcrete).
Methodology for stability analysis of entry-type underground excavations

L. Burgos, J.A. Vallejos and A. Delonca

2 Methodology

2.1 Scenarios of analysis

The analysis of the entry-type underground excavations stability is realized considering a tunnel span of 4.2 m, which is typical of underground access with the entrance of workers and equipment. The scenarios of analysis defined in this paper depend on different rock mass characteristics and different rock support, presented in the following parts.

2.1.1 Rock Mass

Four critical parameters are considered to evaluate the rock mass stability: (1) the depth of the excavation, \( z \), (2) the ratio of the horizontal and vertical stresses, \( k \), (3) the strength of the rock mass, identified by the uniaxial compressive strength, UCS and (4) the rock mass quality identified by the geological strength index, GSI. Those parameters are chosen according to typical values of Chilean porphyry copper deposit.

Table 1 presents the range of values considered in the scenarios of analysis. 82 tunnels have been defined, which are a combination of the parameters presented. For all cases, the rock mass classification in Q system (Barton 1974) is approached from GSI using the correlation \( GSI = 9 \ln Q' + 44 \). In order to obtain the Q value, the SRF factor was estimated according to the stress of each case.

Table 1 Critical parameter for rock mass

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Deep, ( z ) [m]</td>
<td>250, 500, 1000</td>
</tr>
<tr>
<td>Ratio of the horizontal and vertical stresses, ( k )</td>
<td>1, 1.5, 2</td>
</tr>
<tr>
<td>Uniaxial strength compression, UCS [MPa]</td>
<td>50, 100, 150</td>
</tr>
<tr>
<td>Geological strength index, GSI</td>
<td>Poor-Regular-Good</td>
</tr>
</tbody>
</table>

2.1.2 Support system

Two support systems are used in the analysis: (1) the rock bolts and (2) the layers of shotcrete. These two systems are mainly used on their own or combined in the underground tunnels. Moreover, they are easy to represent in a numerical modeling analysis. The design of support has been estimated using the empirical support guidelines of Barton (Barton 2002). These guidelines defined nine categories of support, the function of the equivalent dimension, \( De \), and of the rock mass quality, expressed by the Q parameter (Barton et al., 1974). The equivalent dimension corresponds to the ratio between the width and the degree of instability of an excavation, expressed as the excavation support ratio, ESR. This value is assumed to be equal to 1 and constant for all the scenarios to avoid its influence in the proposed analysis.

The first support system presented is rock bolts. The type of rock bolt considered in the analysis is the grouted bolts. This support is based on the contact between the bolt and the walls of the borehole using resin anchors injected along the element (Stillborg 1993). The key parameters of this type of support are the length and the spacing of the elements as well as the resistance of both the steel and the grout (Hoek et al., 1995). The length of the elements is evaluated by the Barton equation (Barton et al., 1974) and is equal to 2.6 meters. The spacing of the elements is described considering the guidelines of Barton (Figure 1), which defined two zones. The zone (b) presents a rock mass quality from fair to very good, highlighted by a Q higher than 4. It thus does not require a support (pattern bolt). In the case of the zone (a), for a fixed equivalent dimension, \( De \), the dimension spacing between elements will vary between 1 and 1.5 meters. The rock mass quality is comprised between extremely poor and poor.
Figure 1 Zones for rock-bolts design. After Barton 1974.

The bolt properties considered in the analysis are typical for steel bolt: capacity of 18 MN diameter of 20 mm and young's modulus elasticity of 200 GPa. The analysis of the interaction between the bolts and the rock mass is based on the interface bolt-rock mass. It does not consider the interaction bolt-grout-rock mass.

The disposition of the support elements surrounding the excavation is presented Figure 2. The width of the excavation is equal to 4.2 meters, and the length of the bolts is equal to 2.6 meters. The position of the elements of support is defined from the center of the gallery to the exterior.

Figure 2 Disposition of the support elements

The second type of support presented is shotcrete. Based on the empirical method of Barton (Barton et al., 1974), a mix between different reinforced and non-reinforced shotcrete is used in the analysis, depending on the support request. Figure 3 presents the three zones for designing the reinforcing layer of shotcrete. The zone (a) defines the use of reinforced shotcrete. The thickness of the reinforced shotcrete ranges from 50 to 150 millimeters, because applied on extremely poor and very poor quality of rock mass (Q<1). The zone (b) defines the use of shotcrete, without considering reinforcement. The thickness of the shotcrete ranges between 40 to 50 millimeters, and the quality of the rock mass is poor. Finally, the zone (c) no defines the use of support.
The strength properties of the material are evaluated based on the values presented in the literature (Mahar et al., 1975; Saw et al., 2009; Carranza 2014). The Table 2 summarizes the strength properties of the unreinforced shotcrete and of the steel fibre reinforced shotcrete with 30%.

Table 2  Strength properties of unreinforced and reinforced shotcrete

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Unreinforced shotcrete</th>
<th>Steel fibre reinforced shotcrete with 30%</th>
</tr>
</thead>
<tbody>
<tr>
<td>Elastic modulus, [GPa]</td>
<td>8</td>
<td>13</td>
</tr>
<tr>
<td>Poisson's ratio, ν</td>
<td>0.2</td>
<td>0.2</td>
</tr>
<tr>
<td>Unconfined compressive strength, [MPa]</td>
<td>30</td>
<td>40</td>
</tr>
<tr>
<td>Tensile strength, [MPa]</td>
<td>4</td>
<td>5</td>
</tr>
</tbody>
</table>

2.2  Shear strength reduction method

The shear strength reduction method, SSR, is a method mainly used to determine safety factor through the progressive reduction of strength parameters of the material, bringing the model to a state of equilibrium (Dawson et al., 1999). The method is based on the Mohr-Coulomb failure criterion. It allows the assignment of safety factors from a systematic reduction of the strength properties of the material to induce failure. The shear strength ($\tau$) is compound by a cohesion (c) and a friction dependent of the normal stress ($\sigma_n \tan \varphi$) (Equation (1)).

$$\tau = c + (\sigma_n \tan \varphi) \quad (1)$$

The shear strength reduction method is expressed by the equation (2).

$$\frac{\tau}{SSR} = \frac{c}{SSR} + \left(\frac{\sigma_n \tan \varphi}{SSR}\right) \quad (2)$$

In this case, the cohesion and friction factors can be represented as (Equation (3)): 
Thus, the SSR values gradually increase until obtaining the safety factor of the model.

To date, the SSR method is used only to evaluate the slope stability. However, this method can be used to define a criterion able to integrate aspects of both rock mass and support elements, as presented in this paper. Recent studies suggest the use of different approaches to applying SSR method, based on Hoek-Brown criterion (Hammah et al., 2005; Hoek et al., 2002; Fu and Liao, 2010; Chakraborti et al., 2012). It allows the behavior of the rock mass to be described in a better way.

2.3 Numerical Modeling

2.3.1 Finite Element Method- FEM

The Finite Element Method, FEM, is a numerical method allowing the realization of simple and reliable calculations with shorter analysis. It consists of dividing the continuous subsets of elements connected together through a series of points. The elements interact with each other and establish the equations that govern the behavior of the model, which give approximate solutions (Bull 2003).

The models realized in the proposed analysis are two-dimensional models. They have been simulated using the software RS2 of Rocscience (Rocscience 2015). This software allows the interaction between the rock mass and the support, and the SSR method to be taking into consideration. The models are considered to be elastoplastic, isotropic, without water and discontinuities, and in plane strain. The elastic properties of the material are evaluated from the literature (Hoek et al., 1995; Stacey and Page 1986), and are representative of the mining in Chile.

The bolts have been simulated through the Fully Bonded formula proposed by the software. The bolt is divided into elements according to the intercept mesh of the finite element. The elements do not interact with each other. The only interaction of one element with the other one is indirect, based on the modification of the rock mass. The elements fail by tension. Thus, the failure of one element does not cause the failure of its closer neighbors.

The shotcrete has been simulated through a Liner of the type Standard Beam. Its representation considers a combination of axial load stresses and bending. Within the analysis are considered elastic properties (elastic modulus and Poisson's ratio), and the option of working with plastic or elastic materials is given, so strength parameters which vary, depending if shotcrete is required with or without reinforcement.

2.3.2 SSR implementation

The SSR method is implemented to the software RS2 considering as criteria the maximum displacement. Thus, the SSR method implemented as such can be applied on slopes. However, this approach will not allow to study the stability of an excavation considering support. In order to implement it in an adequate way, the following steps are defined:

- **Inputs of the numerical model**: the input data of each model are defined based on the scenarios of analysis, considering the variation of the rock mass characteristics and the support design. The model is defined in the finite element software;

- **Definition of the basic case**: in this first case, the SSR value is taking equal to 1. Then, the stability of the model is evaluated considering the proposed stability criterion. Note that the proposed stability criterion is defined further. If the model is in failure, the proposed stability criterion is taking equal to 1. If the model is stable, the rock mass properties and the rock support parameters are reduced.
• **Incrementing of the reduction factor and re-adjustment of the properties:** in the case of a stable model, the rock mass properties and the rock support parameters are reduced. This reduction is done considering the same magnitude for the strength material properties and for the system support (Dawson et al., 1999). The support strength properties of the bolt and shotcrete are normalized by the value of the increased factor, which has been done manually. The calculated data are entered manually into different files according to the value of the reduction factor evaluated.

• **Proposed stability criterion:** at each incrementing, the stability of the model is evaluated considering different values of SSR. When the model is unstable (according to the depth of damage and failure of support elements), the critical value of SSR is equal to the value for stability criterion for this specific case.

The software RS2 allows, in its post-processing module, the generation of approximations from the display and interpretation of results of stresses and displacements. One of the most relevant analytical tools in the analysis of tunnels are the elements in failure, which represent the number of elements which have failed either cut or tension. Their analysis allow the establishment of the degree of failure, in the outline of an excavation and support system. The analysis of the bolt element and shotcrete can be performed by the observation of the axial load, the failure portion elements and graphic bearing capacity among others.

3 Results

In order to describe the results and the stability criterion, one of the 82 combinations between rock mass parameter and support design will be using bellow. This case represents an excavation at a depth of 250 m, with a ratio of the horizontal and vertical stresses equal to 1. According to Q-System, the rock mass of the case study is classified as very poor. According to the support guidelines of Barton (1974), the excavation should have a bolt pattern with a spacing of 1.2 m and bolt length of 2.6 meters. It also suggests a shotcrete layer of 90 mm thick.

3.1 Stability criterion

The stability criterion is evaluated by the analysis of the results associated with the failure in the post-processing module of the software, both for the rock mass and the supports. One of the essential tools in this analysis is the Yielded Element, whose representation indicating the failure elements in the material. For this analysis, it represents the portion of the rock mass around an excavation that has broken off and is prone to falls or collapses. It generally depends on the orientation of the structures present in the gallery. For practical issue, the Yielded Element is assumed to be the portion of solid elements required to retain, support or reinforce the solid to ensure stability and security in the gallery.

Figure 4 summarizes the representation of the failure mode both for the rock mass and the support elements. The software distinguished the difference between different types of failure: the shear and tension represented with the symbols (x) and (o) respectively. Concerning the elements of support, different symbols are also used to indicate the location and the quantity of failed elements. The shotcrete is represented as a blue dotted line, and the area exceeding the load is red. The patterns bolting are represented as blue, with a green portion of failed elements.
3.1.1. Pattern bolting

Figure 5 presents the extension of the damage zone and the corresponding mode of failure, in relation with the increment of the SSR factor magnitude, for a gallery with a span of 4.2 meters. It is observed, as expected, an increase in the extent of the damage zone as the SSR value increases.

Figure 5 Extension of the damage zone when SSR value is increased

When considering models with higher values than the critical point, a considerable increase is generated in the thickness of failure. It is for this reason that identifying this point is essential to establish a criterion able of evaluating the performance of a support system based on bolts. Moreover failed tension elements represent a small proportion of the total number of elements, and although they have a similar overall performance, they also play an important role in this criterion.

Based on this analysis, it is possible to clearly establish the bolt/rock interaction. When the first fault is generated in any of the elements within the system, this coincides with the portion of the elements of the rock mass that present a tensile failure. The Figure 6 shows graphically this concept, where considering an SSR equal to 1.3, it can be observed the initiation of a failure in one of the support element localized in the roof of the gallery.
3.1.2. Shotcrete

The stability criterion used for the shotcrete had to take into account the behavior of the shotcrete and its interaction with the rock mass. One of the main differences between the behavior presented by shotcrete with and without reinforcement comes from the way in which they support the rock. However, regardless the use of reinforcement, the failure of one specific point occurs perpendicularly to the axial compression plane. This generates traction stresses which lead to the general failure of the reinforcement support of the rock mass.

Figure 7 graphically identifies the critical zone of the analysis for shotcrete. Note that the same dimension of the gallery than in the previous part is assumed: the span of 4.2 meters. This zone is due to the axial load applied on the shotcrete and corresponds to the part of major instability within the excavation. Generally, it corresponds to the highest part of the roof. For most of the scenarios of analysis, it corresponds to a length of 3 meters along the roof.

Based on the analysis of the axial load, the stability criterion determined for the shotcrete corresponds to the SSR value for which occur a major failure in 70% of the elements of shotcrete. This threshold of 70% is evaluated in accordance with the field recommendations and experience of the numerical models made in this study. In Figure 8, it can be observed that the appearance of the first failure on the elements does not indicate a total failure of the system. Indeed, the failures are localized between two patterns bolting and do not have an impact on the stability of all gallery. With the increase of the reduction factor, the zone of failure of the element increases until approximately 76% of elements of shotcrete.
Considering a spacing between the patterns bolting of 1.2 meters:

- 12 elements are in failure for a reduction factor of 1.6, which represents 40% of the length of the layer;
- 76% of the length of the layer is in failure for a reduction factor of 1.8. This value meets the criteria, determining it as the stability criterion for a layer of shotcrete under the evaluated conditions.

This approach is also reflected in practice where cracking of such magnitude in the layer of shotcrete, means taking mitigation and sanitation extra fortification system processes involving wedging and removal of the layer of shotcrete to repair the damaged parts.

### 3.2 Stability criterion of the entire system

Once evaluated both stability criterion for the support elements (bolts and shotcrete), it is required to establish a global safety criterion for each scenario considering a criterion able to evaluate in conjunction the bolt and the shotcrete. Table 3 summarizes the proportion of elements in failure, the length of the failure and the stability criterion associated with the same gallery presenting a span of 4.2 meters, considering the bolts and the shotcrete. It shows that the bolt presents a safety criterion equal to 1.3 and only 30% of the elements are failed. Concerning the shotcrete, 70% of elements failed are needed to meet the requirements of the proposed stability criterion (here equal to 1.8). This indicates that the bolts are critical support elements, and are considered as the limit parameter, relative to the support given to the rock mass.

<table>
<thead>
<tr>
<th>Support</th>
<th>Yielded Element</th>
<th>Extension of the damage zone</th>
<th>Stability Criterion</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rock-bolts</td>
<td>30 %</td>
<td>0.4 m</td>
<td>1.3</td>
</tr>
<tr>
<td>Shotcrete</td>
<td>76%</td>
<td>1.8 m</td>
<td>1.8</td>
</tr>
</tbody>
</table>

It is thus possible to establish a quantitative method to evaluate the stability of the underground excavations which evaluates the performance of support systems with different characteristics and together. Figure 9 summarizes the proposed methodology.
Conclusion

The initial postulate of the Shear Strength Reduction (SSR) method (Dawson et al., 1999) suggests an analysis based on the displacement of elements, to determine the limit slope failure. The application of this method in tunnels involves various modifications. However, the SSR method presents great advantages over others. Among others, the SSR method is quick to use. Moreover, it allows the support strength parameters to be integrated into a stability analysis at early stage of engineering. The consideration of the SSR method into the study presented in this paper leads to evaluate an appropriate stability criterion.

Thus, the analysis of yielded in support elements allows establishing a relation between the support design and rock mass parameters that can be used as a criterion for determining stability in the underground entry-type excavation. In this way, it’s possible to propose an empirical methodology that integrates both analytical and numerical modeling tools.

In this study, limitations in the numerical modelling of support are identified. Differences are observed between the model results and field observations of rock bolts, for example, load capacity, failed elements, interaction with other reinforcement (shotcrete), among others.

In spite of the encouraging results of this study, they have to be validated on real cases to refine the method, and ensure the common use of the method.
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Mining in extreme squeezing conditions at the Henty mine

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Abstract

Squeezing conditions in hard rock underground mines with defined foliation/schistosity/bedding fabric are becoming more common as mines develop deeper. Often mines are finding themselves in squeezing ground without having planned to deal with these challenging conditions and can make time consuming, costly and hazardous mining decisions to keep ore drives serviceable. An understanding of what is occurring around the underground opening is critical and implementing practical yet proven ground support strategies is essential.

The Henty Gold Mine, located in western Tasmania, Australia has recent mining experience in extreme hard rock squeezing conditions. Examples of monitoring data collected at Henty Mine to manage and control squeezing ground conditions are presented. A mining drive closure classification more suited to extreme closure has been produced. The ground support methods are described and the design process used to justify the ground support methods using advanced numerical modelling techniques are discussed.

1 Introduction

The Henty Mine is located in western Tasmania, Australia and has produced gold since 1998, see Figure 1. A number of orebodies have been mined from near surface to a maximum depth of about 900m below surface for a total production of 4,690,789 tonnes at 9.1 g/t Au for 1,370,285 ounce Au (end November 2015). Production rates, mining and haulage methods have varied over the years, depending on the characteristics of the orebody being mined. Shaft haulage of ore was ceased in 2010 in preference to decline truck haulage and the paste plant was decommissioned in March 2008 due to the remaining orebodies being narrow and mined by longitudinal methods, rather than the transverse stoping methods that were used in the wider areas of the Darwin and Darwin South orebodies.

![Figure 1 Location of Henty mine, Tasmania, Australia](image)

During 2014 and 2015, three stoping areas were in production, being the Read Zone (overhand cut and fill), the Darwin South Remnant Zone (longitudinal open stoping) and the Newton Zone (longitudinal open stoping).

Squeezing ground was not considered during the planning of ground support design of the Newton Zone due to side wall strains of less than 2% experienced in most Henty Mine development headings and declines. Mining conditions changed with the development of ore drives experiencing extreme squeezing in the Newton Zone.
1.1 The newton zone

a) The Newton orebody is located between 650m and 800m below surface level with an approximate vertical dip and striking north-south. There are two parallel ore lenses, each about 3.5m width, with parts of each ore lens sub-economic for open stoping. The distance between the two ore lenses varies between about 5m and 8m, refer to Newton Zone Longitudinal Section Figure 2.

Mining development was based on a 20m level spacing and ore drive development was initially 3.6m wide, 4m high arched profile. The development was later changed to 4m width when the extent of squeezing conditions became apparent with ongoing development experience. The Newton Zone stope strike lengths were generally around 17m and stopes filled with rock fill. Narrow rib pillars were left behind to contain the rock fill.

b) Henty Mine Longitudinal Section

c) Newton Zone Longitudinal Section

Figure 2  a) Henty mine longitudinal section  b) Newton zone longitudinal section
1.2 Geological setting

The Henty Mine local geology is Late Cambrian Volcanogenic Hosted Massive Sulphide (VHMS) deposit that has been subsequently altered and overprinted by late stage hydrothermal and structural events.

The Newton Zone orebody at the Henty Mine is hosted within rocks locally referred to as Volcanics, with variable amounts of Sericite alteration. The Sericite alteration gives the rock a greenish appearance, is lower strength than unaltered rock and has a pronounced foliation intensity. Foliation dip varies between 75° and vertical. Foliation spacing is greater than 20 cm in the Henty Mine country rock but foliation spacing decreases significantly in the Newton Zone to as little as 5 mm to 10 mm spacing. Foliation is oriented parallel to the Newton Zone orebody and is present in both the orebody and the country rock.

Rockmass strength of the Newton Zone is influenced by the anisotropic conditions. Rock strength testing at orientations perpendicular/parallel and at about 45° to the foliation display a strength ratio of 8:1, meaning intact sample strength is 8 times stronger parallel to the foliation.

The principal stress orientation at Henty Mine is approximately horizontal and oriented east-west, perpendicular to the strike of the Newton orebody and is highly deviatoric with the ratio of σ1:σ3 measured at 3.6.

The conditions causing the extreme squeezing in the Newton Zone are described in Figure 3 and are:

- A prominent foliation feature is present with close spacing.
- Ore drive development is parallel to the foliation.
- Alteration decreases the rock strength.
- The principal stress is oriented perpendicular to the foliation.
- There are parallel ore drives. The parallel ore drives create an unconfined pillar between the two ore drives which exacerbates wall squeezing.

Figure 3 Contributing conditions for the development of squeezing in anisotropic rock

2 Description of squeezing conditions at Henty mine

Mine development at the Henty Mine has generally displayed less than 2% strain and in most cases mine openings do not require rehabilitation or monitoring with instrumentation during drive service life. When mining in the Newton Zone strains in excess of 50% have been recorded.
2.1 Practical definition of strain

Drive closure in squeezing ground is usually described by the amount of strain experienced between the points of maximum closure since the drive was first developed, as has been described clearly by Potvin and Hadjigeorgiou 2008. For practical reasons many mines, including Henty, monitor the drive closure between the walls at 1.5 m height above the floor. This is an accessible monitoring height, monitors the squeezing where the majority of movement is occurring (in the walls due to the near vertical foliation) and provides mine management with information regarding squeezing rate for mechanised machinery access planning.

Drive closure is practically defined as the squeezing distance divided by the original drive measurement, or as:

\[
\text{Drive Closure (Strain %)} = \left(\frac{A - B}{A}\right) \times 100
\]  

(1)

Where:

- \(A\) = Original drive width.
- \(B\) = Current drive width.

When describing drive closure in areas where the drive walls have been stripped once or more, the strain should be described against the original drive width, or as:

\[
\text{Post Stripping Drive Closure (Strain %)} = \left(\frac{(A - C) + (D - B)}{A}\right) \times 100
\]  

(2)

Where:

- \(C\) = Drive width before stripping.
- \(D\) = Immediate post stripping drive width.

The stability performance of the roof is associated with the total drive strain and it is therefore critical to describe the drive strain as a sum of the strain both before and after drive stripping, as shown in equation 2.

Mine management commonly require estimates of when stripping is expected and advanced warning of additional ground support installations. Estimates can be made from the drive closure monitoring results and by interpreting the results against previous mining case studies in squeezing ground.

2.2 Newton zone monitoring results

Monitoring of the Newton drive closure at Henty Mine was completed by methods including survey of points along each ore drive (drive closure monitoring), survey monitoring of points on the roof at intersections and with numerous extensometers installed along the drives in both the walls and roof (in excess of 30 extensometers installed in the Newton ore drives). Monitoring within the Newton Zone was an important part of the ground support design verification process.

Visual observation of drive performance commonly included shearing of foliation in the drive side walls leading to offsets and ‘guillotining’ of the rock bolts and significant drive wall closure at up to 20 mm/day but more commonly between 2 to 4 mm/day. The roof and floor bulked and heaved respectively, refer to Figure 4.
The drive closure monitoring data shown in Figure 5 describes the wall movement over time in the NW1765HWN, an ore drive positioned parallel to foliation and with a footwall drive located in close proximity to the east. This wall movement is typical of ore drives within the Newton zone. Wall strains of over 30% were experienced with only minor regression in the wall closure rate. Small adjustments have been made to smooth steps in the monitoring data, as stripping did take place along the entire length of the ore drive causing unavoidable loss of monitoring points and short term restriction of access to the drive preventing continuous drive closure survey.

Extensometer monitoring instruments were installed throughout the Newton Zone. The majority of extensometers were installed in the roof, to verify any longer term roof bulking and loosening. Over time,
monitoring within the walls using extensometers became less of a focus to mine management due to the following:

- The design model was verified and showed unravelling from the base of the wall was the likely failure mechanism.
- It was demonstrated through practice that the walls were stable with support and the monitoring results were predictable and consistent.
- In many cases, the drive walls were stripped with new reinforcement and support installed to allow for ongoing machinery access. This alleviated the risk of wall failure initiated by unravelling or toppling.

Monitoring data from within the walls and roof of the ore drives demonstrated consistent patterns. The first 2m of wall material showed about 80% of the total side wall strain, as shown within Figure 6. Deeper within each wall the yield zone continues but at reducing strain amounts. Between 2m and 5m depth about 10% of total strain, between 5m and 7m about 6% of total strain and between 7m and 10m about 4% of total strain.

![Figure 6 Typical sidewall behaviour](image)

The ore drive roofs performed consistently and predictably, despite progressively bulking over time. The majority of ore drive roof strain occurred between 3m and 6m depth, beyond the depth of the 2.4m long roof bolts. No increase in the rate of roof bulking relative to drive closure rate was noted, even at drive closure strains beyond 20%.

Induced stress in the roof clamped the roof material in place, until roof bulking caused eventual breakup of up to 1m of roof material at the excavation boundary. This break up of material at the roof boundary occurred only when drive closure strain had well exceeded 20%. This highlights how time-dependant rock mass deterioration must be understood during ground support design and verified during mining.
2.3 Newton zone intersections

Ore drive intersections experienced more pronounced roof bulking than compared to the ore drives. The geometry of the intersections was a significant contributing factor to the roof bulking, with the ore drive access allowing for more pronounced wall squeezing in the ore drive walls adjacent to the access drive.

Cablebolts (fully grouted) in combination with the standard ore drive ground support were used as a ground reinforcement method to maintain stable conditions at deforming intersections. With detailed monitoring, Henty Mine extended the life of intersections without the need for excessive ground support rehabilitation.

The serviceability of intersection cablebolts was limited to 120 mm of roof movement monitored by conventional survey techniques. This was based on the possible yield of the cablebolt strand beyond 120 mm of movement and that a cablebolt stretches over an assumed length due to debonding at the cable/grout interface with certain amounts of cable anchor pull out and wedge pull in at the plate.

Intersection roof bulking exceeded hundreds of millimetres in multiple Newton Zone intersection locations and all were maintained as safe and stable accesses for mining operations without the need for excessive rehabilitation campaigns. Wall cablebolts were critical for the support of ore drive intersections when the intersection was required to be serviceable for periods of greater than 18 months.

Monitoring of intersection roof bulking in Newton Zone suggests that the majority of roof movement was occurring beyond the cablebolt embedment length, which were 6m long in most instances. Cablebolting was successful in reinforcing the 6m above the intersection and holding the material together while the ongoing movement of the walls allowed the “6m reinforced block” to move into the intersection. This deeper roof movement may also be related to shearing on foliation planes and further work modelling this mechanism is required to validate these observations. Extensometers were essential in the daily management of the intersections, refer to Figure 8.
3 Closure strain classification for extreme drive squeezing

Definition of the squeezing amounts required to cause problems to mine operators is misleading in most of the literature produced to discuss squeezing ground conditions. Many squeezing classifications used to describe squeezing ground were developed for civil tunnels e.g. Hoek 2001, Aydan et al 1993 and Singh et al 2007. Civil tunnels commonly install temporary stiff liners at a set distance from the excavated face and permanent liners within days or months of the temporary ground support being installed and this is distinctly different to the development of a temporary mining opening. Tunnelling classifications of squeezing ground are applicable to mining operations that are continuing to use fibre reinforced shotcrete as a liner, although many mine operations in Australia have discontinued this practice, some are still persisting with the use of fibre reinforced shotcrete in squeezing conditions.
The position of the yielding material at the excavation boundary is important in the classification of squeezing severity. Observations and experiences from the Henty Mine are based on a near vertical foliation with the majority of yield within the walls. In mines with anisotropy that positions the zone of greatest strain at the shoulder or roof of the tunnel, the amount of strain defined as being manageable to operations will be less, due to the walls being inherently more stable than the shoulders or roof.

An example of manageable strain within the Newton Zone 1785HWS ore drive, developed parallel to foliation is shown in Figure 9. This drive was one of the last developed in the Newton Zone and was better planned, supported and reinforced to avoid stripping of the drive prior to stoping commencing. The drive experienced strain of up to 10%. The visual change over time of what 10% strain actually looks like is shown in Figure 10 (this corresponds with up to about 400 mm of drive closure at this location). It should also be acknowledged that drives with identical amounts of strain will perform differently in terms of stability.

![Graph showing closure monitoring data and observations of performance at the 1785HWS](image)

**Figure 9** Closure monitoring data with observations of performance at the 1785HWS

![Photographs of 1785HWS](image)

**Figure 10** Photographs of 1785HWS. Visual indications of drive change are relatively minor, despite the 10% strain being associated with about 400 mm of closure.
The experience of detailed closure monitoring and observation of the Newton Zone has been summarised in Table 1. This classification of closure strain acknowledges that wall stripping cannot be avoided in some circumstances.

**Table 1  Closure strain classification for mining parallel to a vertical anisotropy orientation**

<table>
<thead>
<tr>
<th>Drive Strain ($\varepsilon_t$)</th>
<th>Mine Classification</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>$0 &lt; \varepsilon_t &lt; 2%$</td>
<td>Few Support Problems</td>
<td>Generally no significant deterioration of the drive.</td>
</tr>
<tr>
<td>$2% &lt; \varepsilon_t &lt; 5%$</td>
<td>Light Squeezing</td>
<td>Squeezing is now noticeable at the shoulder. Ground support (with no stiff liner) requires no significant rehabilitation.</td>
</tr>
<tr>
<td>$5% &lt; \varepsilon_t &lt; 10%$</td>
<td>Fair Squeezing</td>
<td>Minor requirement for replacement of failed wall friction bolts (less than 5% of wall bolts). No deterioration of the roof observed.</td>
</tr>
<tr>
<td>$10% &lt; \varepsilon_t &lt; 20%$</td>
<td>Stripping Zone</td>
<td>Generally reaching the limit in terms of reliable bolt performance, wall stability and width requirements for machinery access. Stripping required.</td>
</tr>
<tr>
<td>$20% &lt; \varepsilon_t &lt; 40%$</td>
<td>Post Stripping Zone</td>
<td>This essentially means that the drive has been made safe with the removal of unstable material, and re-installation of pattern ground support. The roof requires possible re-support as the material begins to break up close to the excavation boundary. Bolt warping and breakage common in the shoulders and possibly in the roof.</td>
</tr>
</tbody>
</table>

### 4 Ground support methods

Ground support in Newton Zone was designed to prevent wall failure initiated by unravelling from the base of the wall and to reinforce the roof loosened zone at the excavation boundary. Induced stresses locked the foliation structures together in the roof, meaning a relatively light ground support regime was sufficient. The walls remained stable with relatively light bolting, with flexible surface containment being most critical for stability.

Within the Newton Zone ore drives, walls were meshed floor to floor with 5.6 mm wire, 100 mm aperture galvanised mesh. The mesh was pinned with galvanised 47 mm friction bolts, 1.8m long in the walls, 2.4m long in the roof, on a 1.2 m x 1.2 m pattern. Two rows of 8 mm mesh straps were installed running along each wall to provide resistance to mesh join splitting and mesh wire strand breakage at higher strains. Cablebolts (fully grouted, 6m long twin strand and bulbed) were installed selectively at stope brow positions, where narrow pillars existed between parallel ore drives or where side wall squeezing needed to be slowed to prevent further drive stripping prior to stoping.

Other strap types were used, mainly to use up ground support stock at site. Through experience the mine developed a preference for using an 8 mm mesh strap, rather than a 1.9 mm thick w-strap. The w-strap was found to tear under load and was more susceptible to loader bucket damage.

For the ground support system to be a success in squeezing ground conditions, the following items are considered to be essential components:

- Rehabilitation of all squeezing drives is a very regular task, to replace failed bolts or torn mesh and address all issues prior to the development of a potential ground control hazard.
• Friction bolts require modification to ensure all bolt components match the friction bolt tube UTS of 18 t, as the bolts became locked in place within the walls due to shearing on foliation structures. With no consideration of the friction bolt surface connection, the bolt ring and in some cases the plate will fail at loads possibly below 5 t. This issue is solved by strengthening the surface connection of the friction bolt (both the plate and bolt ring). The plates were upgraded to meet >15 t loading and this was achieved by using a 4 mm thick modified collar plate. The bolt ring, which holds the plate against the wall, more closely resembled a friction bolt pull ring (significantly larger surface connection to the bolt than a 8mm bolt ring) to enable consistent bolt ring failure at >15 t load. Implementing these changes to the friction bolt created a bolt capable of working effectively at up to 200 mm of movement over the bolt length. Due to the foliated conditions and bolt clamping/guillotining in the walls, the full bolt UTS can be utilised.

• Flexible surface support installed to the base of each wall is essential.

Cablebolts (fully grouted and bulbed) achieved between 300 mm and 400 mm of movement over the bolt length when installed into the drive walls. This amount of movement is usually only associated with the performance of debonded cablebolts. Fully grouted cablebolts and progressive breakage at the bolt/grout interface where foliation breaks intersect the bolt (in numerous locations) still allows for some connection of the cablebolt to the rock mass at positions over the entire bolt length.

5 Ground support design for extreme squeezing ground

Considering the complexities involved with the ground conditions at the Newton zone, conventional ground support design methods and numerical modelling techniques are not able to capture the correct failure mechanism. To assist in predicting and understanding of appropriate ground support strategies, a 3D numerical model was developed for the Newton Zone utilising the IUCM method described by Vakili et al (2014). The model accounts for critical factors controlling the failure mechanism in highly stressed and highly anisotropic ground conditions, such as those at Henty Mine. The numerical modelling complimented the ground support design techniques commonly used at Henty Mine that consider the rock mass, rock structures and engineering experience.

This modelling methodology has provided reliable forecast of rock mass behaviour at number of other mine sites with similar ground conditions to Henty Mine, hence the decision to use this approach. The IUCM accounts for mechanisms, such as the transition from brittle to ductile response at various levels of confinement, dilatational response, strength anisotropy, and
Figure 11, this modelling method was able to forecast the rock mass response to mining with a high level of precision using the realistic input parameters obtained from lab testing and field investigations. A good correlation between monitoring data and model outputs was achieved. This model was used for optimisation of ground support design, primarily at stope brow positions and proved to provide a reliable and cost effective design method.
6 Conclusions

The Newton Zone at Henty Mine was mined successfully without a ground control incident (rock fall or failure), even though the ore drives experienced strain described as being extreme. The Newton Zone ore drives experienced as much or more strain than other published squeezing ground case study locations in Canada and Australia. Monitoring data, rather than just visual observations was used to confirm the nature of the movement around the drive openings.

Detailed monitoring has confirmed the depths and extent of the movement zone around the drive. The relationship between strain in the roof and the walls was presented with no increase in the rate of roof bulking relative to drive closure rate, even at drive closure strains beyond 20%.

It was demonstrated that installed ground support can be relatively light in squeezing drives and still maintain safe and stable openings for personnel. Friction bolts with mesh and straps to floor retained the walls and friction bolts were modified to strengthen the surface connection of the bolt which extended bolt service life and reduced the need for rehabilitation of failed bolts. Intersections were maintained in serviceable condition, despite hundreds of millimetres of roof bulking and without excessive rehabilitation through better understanding of the mechanism of movement.

Ground support design for extreme squeezing conditions is a complex and difficult task. A 3D numerical model using the IUCM method assisted by predicting and understanding appropriate ground support strategies. A good correlation between monitoring data and model outputs was demonstrated to provide reliable forecast of rock mass behaviour.
Acknowledgement

Thanks to the owner of Henty Mine, Unity Mining and specifically Rob McLean, for permission to publish this paper. Thanks to Danny Morrison (Mine Superintendent), Mike McCracken (Mine Manager), Peter Wylie (Mine Surveyor), Scott Jones (General Manager) and the operations crews, who supported the collection of monitoring data, completed frequent visual inspections and helped spread the message that safe mining in extreme squeezing conditions is achievable.

References

Numerical modelling of dynamic response of underground openings under blasting based on field tests

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Abstract

In order to assess the capacity of ground support systems when subjected to dynamic loading, simulated rockburst tests by using blasting have been conducted at LKAB Kirunavaara underground mine. In this paper, a numerical simulation for one of the field tests is conducted using LS-DYNA code to numerically investigate the effect of the different aspects of the charge design including the initiation point and the geometry on the test results. In the simulation, an explosive material model is used to model the detonation of explosive used in field tests and the Riedel-Hiermaier-Thoma (RHT) material model is used to model the dynamic response of the rock mass. The decoupling effect between the explosive and the wall of borehole is also taken into account in the model. The numerical results show a similar particle vibration pattern and a crack pattern to those of the field measurement. The effects of the position of the initiation point and the charge structure on the dynamic response of rock mass are also discussed. The results can be a reference for blast design for future field tests.

1 Introduction

Rockburst risk is an increasing problem in the underground mining worldwide, as the general trend is for mines to operate in deeper environments. In most mines affected by seismicity, the first line of defence to mitigate the potential consequences of rockburst is to install dynamic resistant ground support systems (Potvin, 2010). In order to assess the performance of ground support components and systems when submitted to seismic activity and strong ground motion, laboratory tests on core, drop test facilities, simulated rockburst experiments and passive monitoring and back analysis for case studies have been employed for many years in different countries (Hadjigeorgiou and Potvin, 2007).

Drop test and simulated rockbursts by blasting are two popular ways to test and understand the behavior of ground support elements when subjected to dynamic loads. Simulated rockbursts using blasting are generally performed underground in operating mines and they are destructive tests. Although the logistics of setting up and carrying out the tests are complicated and the cost is high compared to drop tests, the advantage is that the ground support is installed and tested in situ and tested as a system rather than individual support elements. Issues such as the interaction with the rock mass and installation procedures are also well simulated and weaknesses in the overall system are highlighted. A lot of simulated rockburst experiments using blasting have been carried out (Ortlepp, 1992; Tannant et al., 1995; Hagan et al., 2001; Espley et al., 2002; Archibald et al., 2003; Andrieux et al., 2005; Heal and Potvin, 2007).

Numerical modelling was used in both the forward planning and the back-analysis of simulated rockburst experiments. Modelling was used to give insight into the design of the experiment, the blast and the positioning of monitoring equipment. Hildyard and Milev (2001) developed a numerical model for seismic wave propagation from the blast to model an artificial rockburst experiment. In their model, the dynamic load consists of applying dilatational pressure ($\sigma_1=\sigma_2=\sigma_3$) along a line of grid-points within the solid material.
of a finite difference mesh. The charge-length and diameter are directly related to the length of the line and the grid-point spacing in the finite difference implementation. Larger diameters are modelled by pressurizing parallel lines or a volume of grid-points. The pressure function describes how the pressure at a point in the source varies with time. The phase of this pressure function varies along the charge line. Minkley (2004) modeled a simulated rockburst using UDEC code. A pressure impulse was used to represent a blasting load in his model. Zhang et al (2013) conducted a back-analyzing for the test results using coupled numerical modeling technique. The blasting is simulated by using finite element method (LS-DYNA) and the dynamic interaction between blasting generated waves and rock mass is simulated by using discrete element modeling (UDEC) with the dynamic input from LS-DYNA (Hallquist 2013). In these studies, the blasting source was represented by an equivalent load form which cannot accurately represent the detonation process of explosive in blastholes and the expansion of detonation products. Especially, only P-waves radiate from the loading boundaries for a two-dimensional model because the velocity of detonation (VoD) in a 2-D model is implied as infinite. However, the investigation of Heelan (1953) indicates that a relatively large amount of the radiated energy from a borehole goes into S-waves, while the rest of it goes into P-waves. It is important to correctly describe the blast load for numerical modelling.

During 2010-2013, a series of underground experiments were conducted at the Kiirunavaara underground iron ore mine which is owned by Luossavaara-Kiirunavaara AB (LKAB) and located in Kiruna, Sweden (Shirzadegan, 2014). The principal objective of the simulated rockburst experiments has been to assess the in-situ performance of different ground support systems under dynamic loading. It is surprising that very high peak particle vibration velocity (PPV) (7.5 m/s) near the sidewall surface were obtained, but with little damage to the support system. With the gradual increase of explosive charge and slight adjustment of burden, the whole tested panel was then fully destroyed. Why it causes such totally different results is not clear yet and most importantly how to design blast in order to effectively investigate the support systems become extremely tough.

In this paper, Test 2 in Shirzadegan (2014) is numerically investigated using the LS-DYNA code. The blast load is directly modeled with an explosive model in LS-DYNA. The crack pattern and the particle vibration velocity are compared to the experimental results.

2 Numerical model

2.1 Descriptions of the model

The southern wall of the cross-cut 93 located in block 9 mining level 741 m was selected to conduct Test 2. The size of the cross-cut was 7.0 m \((W_{cc})\) in width and 5.2 m \((H_{cc})\) in height and the width of the adjacent pillar was approximately 18m, see Figure 1. Rock types in the tested area comprised ‘syenite porphyries’ (mainly trachytes to trachyandesites) of variable character. The blasthole was drilled from the adjacent footwall drift in a direction parallel to the crosscut and was around 15 m long. The diameter of the blasthole was 152mm and the average distance (burden) of the blasthole to the test wall was 3.9m. The height of the blasthole to the floor \((H_{bh})\) was 1.6 m. Two different charge concentrations, each one around 5 m in length, were used inside the blasthole for generating different dynamic loads on the panel in one blast. The charge diameter of the high and low charge segments were 98 mm and 76 mm respectively, indicating to a decoupled charge structure with different decoupling ratios. The length of each charge segment was 5 m. The blasthole was toe primed and was left unstemmed to vent the detonation gas and further reduce the effect of detonation gas. The used explosive was NSP711. The reasons for selecting this explosive were the lower amount of gas production compared to other commercial explosives, high VoD and blasthole pressure resulting in getting more energy through shock wave than the gas expansion (Zhang et al. 2013).

According to the in-situ test, a numerical model was generated with Truegrid software (Rainsberger 2006), see Figure 2. The reinforcement at the tunnel wall is not considered in this model. The difference between the numerical model and the field experiment is that the blasthole is located at the left side of the drift in
the numerical model. This model consists of approximate 15 million hexahedral elements and the element size is roughly 5cm. A Massively Parallel Processing (MPP) version LS-DYNA solver was used to run this case. The numerical model was divided into several blocks and each part was represented in different colors, which is for the convenience to select the nodes which correspond to the locations of the accelerometers installed. All blocks which represent the rocks have exactly the same parameters.

![Figure 1](image1.png)

Figure 1  Schematic diagram of test layout and blast design. (a) Top view, (b) front view and (c) photo of instrumented side wall in Test 2

![Figure 2](image2.png)

Figure 2 LS-DYNA model

### 2.2 Material properties for modelling

The NSP 711 explosive used in the field test is modeled with an explosive material model in LS-DYNA and with the Jones-Wilkins-Lee (JWL) equation of state (EoS) (Lee et al., 1968) as Eq. (1).

\[
p = A \left( 1 - \frac{w}{RV} \right) e^{-RV} + B \left( 1 - \frac{w}{RV} \right) e^{-RV} + \frac{wE_e}{V}
\]  

(1)
where \( p \) is the pressure; \( A, B, R_1, R_2 \) and \( w \) are constants and \( V \) and \( E_e \) are the specific volume and the internal energy respectively. In eq.(1), \( A, B, \) and \( E_e \) have units of pressure while \( R_1, R_2, \) and \( w \) are unitless.

The parameters of NSP 711 explosive were calibrated by Helte et al. (2006) and listed in Table 1. In Table 1, \( \rho \) is the density of the explosive used, \( D \) is the velocity of detonation of the explosive, \( P_{CJ} \) is the Chapman-Jouguet pressure of the explosive.

### Table 1 Parameters of NSP 711 explosive

<table>
<thead>
<tr>
<th>( \rho ) (kg/m(^3))</th>
<th>( D ) (m/s)</th>
<th>( P_{CJ} ) (GPa)</th>
<th>( A ) (GPa)</th>
<th>( B ) (GPa)</th>
<th>( R_1 )</th>
<th>( R_2 )</th>
<th>( w )</th>
<th>( E_e ) (kJ/cc)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1500</td>
<td>7680</td>
<td>21.15</td>
<td>759.9</td>
<td>12.56</td>
<td>5.1</td>
<td>1.5</td>
<td>0.29</td>
<td>7.05</td>
</tr>
</tbody>
</table>

As mentioned before, the charge structure in the blasthole is decoupling. The gap between the explosive and the wall of the blasthole was filled with air. *MAT_NULL is adopted for air and is combined with a linear polynomial EoS shown in equation (2).

\[
P = \left[ C_0 + C_1 \mu + C_2 \mu^2 + C_3 \mu^3 \right] + \left[ C_4 + C_5 \mu + C_6 \mu^2 \right] E_0
\]

(2)

where \( C_0, C_1, C_2, C_3, C_4, C_5, \) and \( C_6 \) are constants and \( \mu = \frac{\rho}{\rho_0} - 1 \) with \( \rho_0 \) the ratio of current density to initial density, \( E_0 \) is initial internal energy per unit reference volume. For gases which the gamma law EoS applies such as air, therefore eq (2) reduces to \( P = (\gamma - 1) \frac{\rho}{\rho_0} E_0 \). For *MAT_NULL, a pressure \( P_c \) is used to limit the amount of pressure that can be generated by tensile loading. This pressure was set to zero since air does not allow tension. Similarly, since the inertial forces were dominant, the flow was assumed to be inviscid and thus the dynamic viscosity coefficient \( \mu_c \) could be omitted. All the used parameters for air are given in Table 2. \( V_0 \) is the initial relative volume of air in Table 2.

### Table 2 Parameters of air (Olovsson et al. 2003)

<table>
<thead>
<tr>
<th>( \rho ) (kg/m(^3))</th>
<th>( P_c )</th>
<th>( \mu_c )</th>
<th>( C_0-C_3, C_6 )</th>
<th>( C_4 )</th>
<th>( C_5 )</th>
<th>( \gamma )</th>
<th>( E_0 ) (MPa)</th>
<th>( V_0 )</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.29</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0.4</td>
<td>0.4</td>
<td>1.4</td>
<td>0.25</td>
<td>1.0</td>
</tr>
</tbody>
</table>

The rock mass is modeled with the RHT material model in LS-DYNA, which is an advanced plasticity model for brittle materials such as concrete and rock. It was proposed by Riedel et al. (Riedel et al., 1999) for dynamic loading of concrete and implemented in the LS-DYNA code by Borrvall and Riedel (2011). In the RHT model, the description of the stress state is based on the three invariants of the stress tensor for the definition of the elastic limit surface, failure surface and residual strength surface for the crushed material. These three surfaces all are pressure dependent. In this model, the damage is defined using \( D = \sum \frac{\Delta \varepsilon^p}{\varepsilon^f} \), where \( \Delta \varepsilon^p \) is the accumulated plastic strain and \( \varepsilon^f \) is the failure strain. Some of used values for the modeling of the rock are shown in Table3. Here, \( \rho \) is the density of the rock mass, \( E \) is the elastic modulus, \( \sigma_c \) is the uniaxial compressive strength, \( \sigma_t \) is the uniaxial tensile strength and \( v \) is the Poisson’s ratio.

### Table 3 Parameters of rock mass

<table>
<thead>
<tr>
<th>( \rho ) (kg/m(^3))</th>
<th>( E ) (GPa)</th>
<th>( \sigma_c ) (MPa)</th>
<th>( \sigma_t ) (MPa)</th>
<th>( v )</th>
</tr>
</thead>
<tbody>
<tr>
<td>2800</td>
<td>70</td>
<td>180</td>
<td>10</td>
<td>0.27</td>
</tr>
</tbody>
</table>
curve for Accelerometer 9 is plotted in Figure 9 (a) together with the numerical result of a node which corresponds to the position of Accelerometer 9. Also the velocity integrated from the record of accelerometer 18 located 0.75 m behind the surface is compared with numerical modelling and plotted in Figure 9 (b).

In both cases, the PPV from the numerical modelling is lower than that from the field test. One possible reason is that the zone near the surface of the side wall is a fractured zone due to the excavation of the drift by blasting. The investigation of Zhang et al (2015) indicates that the presence of a fractured zone near the surface of the wall can amplify the PPV. It can also be observed that the duration of the velocity-time curve from the numerical modelling is shorter than that from the field test. At the beginning of the vibration velocity curves, the field result and the numerical result have the similar rise rate and the similar waveform.

![Velocity of Accelerometer 9](image1.png)  
(a) Velocity of Accelerometer 9

![Velocity of Accelerometer 18](image2.png)  
(b) Velocity of Accelerometer 18

Figure 3 Vibration velocities at different positions from field test and numerical modelling

### 3.2 Displacement distribution

The displacement distribution at 6 ms on the surface of the side wall is shown in Figure 4 (a) and the displacement distribution at the section behind 0.2m of the surface of the side wall is shown in Figure 4 (b). It can be seen that the peak displacement of the side wall is 2.4 cm and it is 1.75 cm on the section behind 0.2m of the side wall. Displacements from the numerical modelling are smaller than those from the field measurements (Shirzadegan, 2014). One possible reason is that the rock is simulated as continuous material. The numerical results show that the large displacement areas in Figures 4 (a) and 4 (b) are around the join point of the high charge segment and the low charge segment.

![Displacement distribution on the surface of the side wall at 6ms](image3.png)  
(a) Displacement distribution on the surface of the side wall at 6ms
3.3 Crack pattern

After the blasting, the damage distribution in the rock mass is shown in Figure 5 (a). It is hard for finite element method to directly model the initiation and propagation of cracks in rock mass. In this paper, the elements with damage level above 0.7 were blanked out to form cracks in the rock mass after blasting. The crack pattern is shown in Figure 5 (b). The damage on the panel in Test 2 is shown in Figure 5 (c).

It should be noted that the side wall is on the left side of the blast hole in the field test while it is on the right side of the blast hole in the numerical model. So the directions of crack from the field test and the numerical modelling are consistent.

Two cross-sections are chosen in Figure 6 to show the internal cracks. Figure 6 indicates that radial cracks are the dominate crack around the blast hole. The depth of the crack on the surface of the side wall is small. The failure of the side wall of drift is because of the reflection of stress waves.
Figure 5 Damage on the panel from numerical modelling and field test

Figure 6 Internal crack patterns at different cross-sections

4 Discussion

Although using blasting to simulate the effects of rockbursts on ground support systems has been used by many researchers, until now, there is no standard on how to conduct simulated rockburst experiments and different blast designs have been utilized in existing simulated rockburst experiments. The dynamic response of rock mass under blasting depends on several factors such as burden, the amount of charge, the position of initiation point, the charge structure and so on. It is good to know how these factors affect the results for a blast design.

4.1 Effect of the position of initiation point

The low charge segment was close to the open end of the blast hole while the high charge segment was loaded near the toe of the blast hole in Test 2. The detonator is located at the toe of the blast hole. The field measurements indicated that the most of the large PPVs are located in the area which corresponds to the low charge segment (Shirzadegan, 2014). Numerical results also show the similar phenomenon. A lot of factors can affect the dynamic response of rock mass under blasting. The position of the initiation point could be one of the reasons for this phenomenon. To investigate the effect of the position of the initiation point, a case that the initiation point is located at the end of the low charge segment was run.
Two nodes that were located at the surface of the side wall and correspond to the middle of different charge segments are selected to compare their vibration response. Node 11092102 corresponds to the low charge segment while Node 11092203 corresponds to the high charge segment. When the initiation point is located at the toe of the blast hole, the stress wave due to blasting reaches Node 11092203 first and then Node 11092102, see Figure 7 (a). The PPV of Node 11092203 is smaller than that of Node 11092102. When the initiation point is located at the end of the low charge segment, the stress wave reaches Node 11092102 first and then Node 11092203, see Figure 7 (b). The PPV of Node 11092102 is smaller than that of Node 11092203.

According to the numerical modelling, when two different charge concentrations are used inside the blast hole to generate different dynamic loads on the panel in one blast, it could be hard to tell the difference between two dynamic loads from different charge concentrations.

Figure 7 Comparison of vibration velocities for different initiation points
4.2 Effect of charge structure

In Test 2, the high charge segment was located at the bottom of the blast hole. To investigate the effect of charge structure, two cases were investigated numerically. One case is that two charge segments in Section 3 exchange their locations, and the other case is that two charge segments have the same diameter of 98 mm. The initiation point for two cases was located at the toe of the blast hole. The crack patterns of two cases are shown in Figure 8.

Comparison between Figure 8 and Figure 5 (b) shows that different charge structures induce different crack patterns.

The same nodes as those in Section 3 were selected to plot their velocity-time curves. Results are shown in Figure 9. The PPV of node 11092203 in Figure 9 (a) is less than that in Figure 9 (b). It is because that Node 11092203 in Figure 9 (a) corresponds to the low charge segment, which means the PPV is also related to the charge concentration of explosive. Figure 9 (b) also indicates that the vibration response of rock mass is related to the direction of detonation propagation.

(a) Crack pattern after exchanging locations
(b) Crack pattern of same charge diameter

Figure 8 Crack patterns for different charge structures

(a) Two charge segments exchange locations

(a) Two charge segments exchange locations
Numerical modelling of dynamic response of underground openings under blasting based on field tests

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10 | Ground Support 2016, Luleå, Sweden

4.3 Limitations of the numerical modelling

Although LS-DYNA can model the detonation of explosive and the dynamic response of rock mass due to blasting, and the 3-D model can also avoid omitting the effect of shear waves, the spalling of the side wall due to blasting cannot be modelled directly because of the limitations of continuum-based methods.

The ground support was not simulated in this paper. The model will be complicated if the shotcrete and the rebars are added into the model. Zhang et al. (2013) stated that PPVs are not greatly affected when support system is applied, because the shock wave hits the panel rapidly, the support will first move together with the rock mass as a whole without restraining the rock mass markedly before fractures are generated.

The presence of discontinuities in rock mass greatly affects the propagation of stress wave due to blasting. The existing natural discontinuities in rock mass were not taken into account in the numerical model, which could be one of the reasons for the discrepancies regarding velocity and displacement between numerical modelling and field measurements.

5 Conclusion

In order to better understand the results of the simulated rockburst test conducted at Kiirunavaara underground mine, LS-DYNA code has been used to numerically investigate one of the tests. Based on the simulations it can be concluded that:

1) Numerical results indicated to the similar waveform at the beginning of the vibration velocity curve and the similar PPVs distribution to the field measurement. The crack pattern from the field test and the numerical modelling were similar. It could be possible to improve the blast design for the simulated rockburst test by using numerical modelling.

2) The dynamic response of rock mass under blasting strongly depends on the position of the initiation point, the charge structure and the charge concentration. Numerical results show that the PPVs on the tested panel increase along the direction of detonation propagation. It is hard to investigate the effect of different dynamic loads by using different charge concentrations in one blasthole.
References


Optimisation of gateroad support at the depth more than 1000 m in hard coal mines

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S. Prusek, Glowny Instytut Gornictwa, Poland
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A. Wrana, Glowny Instytut Gornictwa, Poland

Abstract

The European countries increasingly carry out underground hard coal mine excavation at depths greater than 1000 m. Such a situation results in the fact that significant vertical and horizontal convergence values occur in the gateroads, which is caused by the impact of considerable loads exerted on the support.

With production being concentrated into even fewer high-performance longwalls, located at greater and greater depth, pressure to ensure the gateroads functionality (required dimensions) continues to increase.

This paper presents the complete process of gateroad support optimisation targeted for given geological and mining conditions. The process consists of the underground investigations on performance of a support system designed by colliery. The performance is described by the gateroad convergence, support load, and shape and height of the fractured zone in rock mass around the longwall gateroad. The results of these investigations constitute the basis for calibration of numerical models. Modelling work is done comprising both, reproducing of rockmass behaviour and gateroad support performance. Once the models are calibrated the work to optimize the existing support systems is undertaken. The following means are considered during optimisation: implementation of additional reinforcements (flexible bolts, stringers etc.), steel parameters up-grading, implementation of heavier V profiles (arches and bolted stringers), change of location of bolted stringers. The main advantages of new developed support systems constitute: increased load bearing capacity; improved stress distribution in particular support system elements achieved by additional reinforcements implementation, reduced unit load exerted on primary support elements (steel arches), better utilisation of load bearing capacity of support scheme elements. The last stage of optimisation process is a validation of the results by underground application and tests of new support systems in analogous conditions as in the first stage of optimisation process.

The real case of gateroad support optimisation carried out in one of Polish hard coal mine under RFCS project (RFCR-CT-2013-00001) titled “Advancing mining support systems to enhance the control of highly stressed ground” is described.

1 Introduction

When applying a longwall extraction system in hard coal seams the proper design of gateroad support plays a crucial role. Proper means such design which ensures that for given mining conditions workings are stable and of proper size during the whole mining process.

In the support design process it is indispensable to assess the technical parameters of the gateroad such as: purpose of the roadway, shape and dimensions, cross-section area, available support in terms of load capacity, yielding behavior or corrosion resistance. Also the determination of rocks
geomechanical parameters and mining conditions including stress state and value and mechanism of load exerted on the support is important (Bigby 2004, Junker et al. 2009, Lawrence 2008, Barczak 2005).

The technical parameters depend directly on the implemented extraction system, thickness of extracted coal seam, air demands, dimensions of mining equipment, expected value of roadway convergence and natural or technical hazard which determine the roadway performance (Szwedzicki 2005, Hucke et al. 2006, Hebblewithe, Lu 2004).

The geotechnical parameters of rock in the roadway proximity as well as the value and direction of stress and load on support depends on the geological structure of rockmass, depth, type, previous and ongoing extraction (Cartwright, Bowler 1999, Colwell, Firth Mark 1999, Mark 1998, Snuparek, Konecny 2010).

Aforementioned parameters constitute input data to empirical and numerical modeling support design codes. Both load exerted on support and load capacity are determined by means of empirical codes (Prusek 2010, Lubosik, Prusek 2010) and by numerical modelling (COSMOS, ANSYS, RS, FACE, FLAC) or authorial (ABC RAMA) codes (Bock, Prusek, Rotkegel 2009, Walentek et al. 2009, Torano et al. 2002, Prusek 2008) while geotechnical parameters and rockmass behavior is determined in laboratory or in situ tests by means of e.g. stiff press, penetrometer, borehole camera, dynamometer, extensometers, strain gouged rockbolt, telltales, overcoring method etc (Bigby, Hurt, MacAndrew 2011, Kukutsch et al. 2013, Bowler, Betts, Altounyan 2008).

In the Polish hard coal mining industry, the gateroads are mostly driven with a 14–16 m² cross-sectional area and the primary support of gateroads is the steel arch yielding support. The support frame is mostly made from the V-shape profile of 24–29 kg/m (up to 36 kg/m) elementary mass, and the frame distance of 0.75–1.0 m predominates. Given that the geological and mining conditions are constantly getting worse, combined support systems including steel arch yielding support and the application of reinforcements become more common. Outby the longwall the support is usually reinforced with friction or wooden props and by bolting the roofbar arches. Inby the longwall the main way to enhance the support and load-bearing capacity is to use wooden props and roadside packs (Prusek et al. 2011). The support parameters as well as type, number and scheme of reinforcements which influence the roadway stability are determined in the support design process.

This paper describes an example of gateroad support optimization for gateroad C-5, where after assessment of rockmass behavior and support performance in the neighboring gateroad C-4, the mechanism of support deformation was determined, new support solution characterized by higher load capacity developed and underground test of that solution performed. Both gateroads were located in similar geological and mining conditions. The measurements started about 200 m before longwall face and were completed on longwall face line. There were no other gateroads in direct vicinity of measurement stations (the gateroad C-4’ was developed after longwall C-3 termination).

2 Assessment of rockmass behaviour and gateroad support performance

2.1 Geologic and mining conditions in the panel of longwall C-3 in seam 404/1

Gateroad C-4 was a maingate for longwall C-3 which was located in seam 404/1 (Figure 1) at the depth of 960 - 1010 m. The seam has a dip of 2 - 12°, directed to the north and north-east.

Longwall C-3 was about 250 m in width, it’s height reached 2.0 m, whereas its average run amounted to about 1050 m.

In the locality of longwall C-3, at a distance of up to 160 m above seam 404/1, coal mining operations were executed in the seams: 401/1, 363, 361 and 360/1. These operations have an effect on the
stress state in the longwall C-3. Beneath the panel of longwall C-3, up to 60 m, no extraction was performed so far.

The roof is composed of: 3.0 m clay shale, 3.5 m sandstone and 9.7 m clay shale while floor is built out of 21.5 m clay shale (Figure 1).

The strength parameters of the coal and surrounding strata are listed in the Table 1.

Table 1  Rockmass strength parameters around gateroad C-4

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Roof strata UCS/UTS</td>
<td>49.61 MPa/3.18 MPa</td>
</tr>
<tr>
<td>Floor strata UCS</td>
<td>23.60 MPa</td>
</tr>
<tr>
<td>Coal seam UCS/UTS</td>
<td>9.35/0.6 MPa</td>
</tr>
</tbody>
</table>

The gateroad C-4 was protected by steel arch yielding support. The basic support parameters are presented in Table 2 and on Figure 2.

Table 2  Gateroad C-4 – support parameters

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Type</td>
<td>ŁP/12/V32/4</td>
</tr>
<tr>
<td>Dimensions (breadth and height)</td>
<td>6.1 m x 4.2 m</td>
</tr>
<tr>
<td>Cross-section</td>
<td>21.8 m²</td>
</tr>
<tr>
<td>Type of steel</td>
<td>S480W (yield stress 480 MPa, ultimate strength 650 MPa)</td>
</tr>
<tr>
<td>Distance between arches</td>
<td>0.75 m</td>
</tr>
<tr>
<td>Reinforcements</td>
<td>flexible bolts</td>
</tr>
</tbody>
</table>
2.2 Underground measurements in gateroad C-4

2.2.1 Methodology of underground measurements

Measurements of rock mass deformations around the gateroad C-4 and load exerted on its support were carried out at measurement stations which were set at a distance of 200 m outby the longwall face (Figure 3).

The following quantities were measured: uniaxial tensile and compressive strength of roof, floor and sidewall by means of the hydraulic penetrometer, vertical convergence and horizontal convergence, roof movement by means of triple height telltales, fissures propagation – by means of the borehole camera, load exerted on the LP-type support sets – by means of hydraulic dynamometers, rockbolts axial load – by means of strain gauged rockbolts.

The measurements were carried out periodically according to the assumed intervals which were dependent on the longwall face advance.

2.2.2 Measurements results

The measurements in the gateroad C-4 were conducted during the period from Nov 2014 to Feb 2015. The results are presented as graphs depicting: change of height and width of the gateroad (Figure 4), values of roof strata displacement and rock mass fracturing zone range around the gateroads (Figure 5), load on the LP support set and distribution of axial forces in the instrumented rock bolt (Figure 6).
The above presented results show that the approach of the advancing longwall face caused that the value of the vertical convergence, after taking into account the floor dinting (up to 0.8m) carried out 150 m before the longwall face, was 732 mm and the horizontal convergence - 271 mm (Figure 4). The underground measurement showed that a phenomenon of floor uplift was crucial in the vertical convergence of the gateroad. It constituted approx. 94% of the total vertical convergence value. This fact is confirmed by the registered readings of the telltales where maximum value of displacement in 10 meters of roof rocks reached 45 mm (Figure 5).

Based on the above-specified results of measurements concerning the gateroads deformation, the minimum value of the cross section area that was within the T-junction was calculated. This is of vital importance in the design of ventilation of the entire area of the longwall, in particular in the case of mining operations within the seams under high methane hazard. The reduction amounted, after inclusion of the floor dinting, 41% (from 21.78m² to 12.92 m²). If there was no floor dinting, the cross section area would be 9.51 m² (reduction by 56%). Such a situation could result in significant difficulties in a proper ventilation of the entire area of the longwall.

The endoscopic tests results (Figure 5) showed that the influence of longwall extraction at great depths did not cause a significant increase of the fracture-zone extent but only an increase of the number of fissures and their total stratification. No changes in the maximum range of roof rock fractured zones between the first and the last measurement were registered, and it was 8.6 m.
However, an increase of the number of fissures, from 11 to 30, was noted. Such a situation is considerable, especially in case of a rock bolt support.

The analysis of measurement results concerning the LP support load carried out using hydraulic dynamometers indicates that the first effects of the abutment pressure are noticeable at a distance of ca. 200 m before the longwall face. From this distance, a systematic increase in the load occurs. The total maximum load of a single LP arch (longwall side and sidewall) set with a spacing of 0.75 m was 272 kN (362 kN/m) (Figure 6). These values did not exceed the admissible maximum load-bearing capacity of support used in the gateroad which was 520 kN/m. Moreover, it can be stated, based on the measured load values, that the support was loaded asymmetrically and, in general, greater load values occurred at the longwall sidewall and the lower values were at the opposite sidewall.

The maximum values of axial forces exceeded the permissible load-bearing capacity of the bolt specified by the manufacturer of 180 kN and reached 270 kN (Figure 6). All the registered load values in anchors constituted tensile forces. According to the studies, the greatest changes in deformation were registered by tensometers located along the bolts that is between 1.45 and 1.9m. Moreover, a significant increase in the value of axial forces in the bolts was recorded when the longwall face was located at a distance of ca. 53 m.

Figure 6 Results of measurements of arch (left) and rockbolt (right) support load for gateroad C-4

3 Numerical modelling: mechanisms of gateroad deformation and support scheme optimization

3.1 Introduction to numerical modelling

The mechanism of gateroad deformation was determined on the base of both: the results of underground measurements carried out in gateroad C-4 and support performance by means of COSMOS/M program (COSMOS/M 1999). In course of numerical calculation in discrete model of arches directional support reactions were compared with the results of underground measurements (e.g. indications of dynamometers installed under sidewall arches).

In order to optimize the support scheme in gateroad C-4 the steel arch support calculations were conducted by applying FEM method in COSMOS/M program. The mechanisms of gateroad deformation (value and distribution of load) and support behaviour model allowed for conducting of the cycle of strength calculations aiming to optimize the schemes of support systems used in considered gateroad. In the numerical modelling the following strength criteria were assumed: - maximum stress in support set elements must be smaller than stress permissible for used steel profiles, - maximum stress in the reinforcement of support sets (steel stringers) must be smaller than stress permissible for profiles, - load of bolts must be smaller than their maximum load capacity.
In the first stage of strength calculations, the model for mapping the construction form of support scheme used in measurement station, has been created with the application of CAD program. Further, using the COSMOS/M program, which calculates the reduced stress according to the Huber-Mises-Hencky hypothesis, corresponding cross-section and material parameters were assigned to particular elements.

Material coefficient $\gamma_s$ and material plastification coefficient $n$ were calculated for the applied sections, which were used for making of particular elements of the support. Their values for steel used for mining supports were assumed in accordance with standard PN-H-93441-1 and PN-H-84042 (Polish Standard 1994, Polish Standard 2004).

Coefficient of plastic reserve of cross-section $m$ is strictly related to the shape of the cross-section of support element-support set arch. It is equal to the ratio of plastic section modulus to the bending section modulus:

$$m = \frac{W_{pl}}{W_x}$$

(6)

The coefficient of plastic section modulus is equal to sum of absolute value of static moment of compression and tension cross-cut area in relation to neutral axis in state of full plastifying.

$$W_{pl} = |S_c| + |S_t|$$

(7)

The permissible stress, determined in the a/m way cannot be exceeded by the reduced stress, calculated - for example - with the application of FEM program:

$$\sigma_{red} \leq \sigma_{dop}$$

(8)

3.2 Numerical modelling of mechanisms of gateroad deformation

The behaviour of gateroad support was reproduced by numerical modelling. The input data are presented above. In the numerical modelling the load distribution have been adapted into the model as long as the results recorded by hydraulic dynamometers during underground measurements were reproduced in the model. The reduced stress distribution according to the Huber-Mises-Hencky hypothesis, in elements of support scheme, displacements of particular nodes, model deformation, values of internal forces, load of elements simulating flexible bolts and colour stress maps were produced during the modelling.

Figure 7 presents support scheme elements in gateroad C-4 (left) and results of reduced stress distribution assessment (right). Table 3 shows comparison of the underground measurements of convergence and support reactions with the results of numerical calculations.
Figure 7  Support scheme (left) and reduced stress distribution in support elements (right) in gateroad C-4

Above presented results of numerical modelling as well as analysis of the geological and mining conditions, implemented support systems and results of measurements allowed to determine the mechanisms of gateroad deformations (Table 3) i.e. distribution of load exerted on support and support movement direction.

Table 3  Mechanisms of gateroad C-4 deformations

<table>
<thead>
<tr>
<th>Gateroad C-4</th>
<th>Legend</th>
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<tbody>
<tr>
<td>Load direction - perpendicular to support</td>
<td></td>
</tr>
<tr>
<td>Load direction - perpendicular to seam</td>
<td></td>
</tr>
<tr>
<td>Support movement direction</td>
<td></td>
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<tr>
<td>, +, -</td>
<td>Values recorded on dynamometers , + higher; , - lower</td>
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</tbody>
</table>

In considered case it was observed and different values of load recorded on the dynamometers placed under support ribside arches the support was loaded asymmetrically and the acting load caused the movements of support frames in direction to the direction of longwall side.

3.3 Numerical modelling of optimised support schemes

In order to develop innovative support systems for gateroad C-4, based on the developed models of rockmass behaviour and support performance, the following means were considered: implementation of additional reinforcements (flexible bolts), steel parameters up-grading, implementation of heavier V profiles (arches and bolted stringers), and change of location of bolted stringers.

The main advantages of newly developed support systems constitute: increased load bearing capacity, improved stress distribution in particular support system elements achieved by additional reinforcements implementation, reduced unit load exerted on primary support elements (steel arches), and better utilisation of load bearing capacity of support scheme elements.

The results of support system optimisation for gateroad C-4 are presented in Table 4.
The study showed that load bearing capacity of innovative support systems can be increased up to about 26% in relation to support systems used by collieries what occurred in optimization scheme no 7 where the position of flexible bolts was changed as well as the profile of the stringer and additional bolts were added at the sidewall. Nevertheless comparison of different optimisation schemes shows that additional bolts or changing the stringer’s profile improves support system bearing capacity just of 1.5 – 4%. Considering the costs of profiles, bolts and more time consuming installation process it was decided to apply support scheme as shown in optimisation stage no. 5 which assumes only the change of stringer position (18% of capacity increase).

4 Underground testing of optimised support system

4.1 Mining conditions at test site and methodology of underground measurements

The gateroad C-5 located close to gateroad C-4 was selected as a testing site of optimised support systems. The gateroad C-5 is headgate of longwall C-4 in seam 404/1 which is neighbouring to panel C-3 where first measurement campaign was carried out (Figure 1). The geological and mining conditions are similar as in longwall C-3. The gateroad C-5 is localised about 20 m deeper than gateroad C-4. Repeated penetrometric test conducted in roof and coal sidewall did not showed relevant deviation in strength parameters.

Longwall C-4 is about 250 m in width, whereas its average panel length amounted to circa 1014 m. The longwall’s height is 2.0 m.

The tested optimised support scheme is shown in Figure 8. The support was arch yielding support type ŁP12/V32/4/A set with a distance of 0.75 m as in C-4 gateroad. Steel arches were supplemented with two rows of long tendon (flexible) bolts installed asymmetrically with steel stringer made of V25 profile.
The arrangement of measurement station in gateroad C-5 was also the same as in gateroad C-4 (Figure 3).

### 4.2 Optimised support system tests results

The measurements in the gateroad C-5 were conducted during the period from Jan 2016 to Mar 2016. The results are presented as graphs depicting: change of height and width of the gateroad (Figure 9), values of roof strata displacement and rock mass fracturing zone range around the gateroads (Figure 10), load on the LP support set and distribution of axial forces in the instrumented rock bolt (Figure 11).

![Gateroad C-5 deformation](image)

**Figure 9** Gateroad C-5 deformation

![Support scheme in gateroad C-5](image)

**Figure 8** The support scheme in gateroad C-5
The above presented results show that the approach of the advancing longwall face caused that the value of the vertical convergence, after taking into account the floor dinting (up to 0.8m) carried out 150 m before the longwall face, was 1725 mm and the horizontal convergence - 650 mm (Figure 9). As in the case of C-4 gateroad a phenomenon of floor uplift was crucial in the vertical convergence of the gateroad. This fact is confirmed by the registered readings of the Telltales where maximum value of displacement measured 50 m outby longwall face, in 10 meters of roof rocks reached 190 mm (Figure 10). About 10 m outby longwall face the total roof strata displacement exceeded measuring scale of used telltales that is 225 mm. In case of gateroad C-5 the reduction of cross-section area amounted, after inclusion of the floor dinting, 30% (from 21.78m$^2$ to 15.2 m$^2$) and with no floor dinting 47% (11.5 m$^2$). Thus the developed support system reduced the gateroad convergence and finally its cross-section area was by 2.3 m$^2$ higher than when original support scheme was implemented. So the convergence expressed in square meters of vertical cross section was by 25% lower.

Comparison of support deformation measurements results of gateroads C-4 (Fig. 4) and C-5 (Fig. 2) it can be stated that optimised support scheme (Fig. 8) allow to reduce the convergence of C-5 gateroad in period from working development till the moment when longwall was at the distance 150 m from measurement station. The convergence value at this period was only 390 mm when in C-4 gateroad it was already 939 mm.

Additionally in gateroad C-5 from 150 m before longwall face the higher value of roof displacement then in gateroad C-4 was observed. This was probably caused by worse quality of lagging and empty space between the support and roof strata. It led to intensification of roof displacement and delayed interaction of standing support with rockmass what was confirmed by the course of load readings on hydraulic dynamometers (Fig. 6 and 11).

The endoscopic tests results (Figure 10) showed that from about 150 m outby longwall face no significant increase of the fracture-zone extent was observed. The boundary of fractured zone in roof strata was 8.5 m (about 150 m from longwall). During last measurement fractured zone extended to about 9.2 m in vertical borehole. Such a situation is considerable, especially in case of a rock bolt support.

The analysis of measurement results concerning the LP support load carried out using hydraulic dynamometers indicates that the first effects of the abutment pressure are noticeable at a distance of ca. 150 m before the longwall face (Figure 11). From this distance, an increase in the load occurs. The total maximum load of a single LP arch (longwall side and sidewall) set with a spacing of 0.75 m was 396 kN (528 kN/m). Highest values of load were registered about 50 m before longwall face, after this measurement the additional friction props were installed in gateroad. Maximal load values...
exceeded the admissible maximum load-bearing capacity of support used in the gateroad which was 520 kN/m.

Figure 11 Results of measurements of arch (left) and rockbolt (right) support load for gateroad C-5

The maximum values of axial forces reached 147 kN (Figure 11). According to the measurements, the greatest changes in deformation were registered by tensometers located along the bolts that is between 1.60 and 1.90 m. Moreover, a significant increase in the value of axial forces in the bolts was recorded when the longwall face was located at a distance of ca. 150 m.

5 Conclusions

In the roadway support design process when the rockmass movements of various intensity (e.g., roof sag, floor heave, and horizontal convergence) as a result of the extraction pressure impact affect support performance coal mine management is responsible for applying appropriate support in the roadways which ensuring stability and work safety for the miners.

The method of roadway support design presented in this paper based on the underground measurements and numerical modelling allows to design economically-friendly solutions of support schemes characterized by higher load bearing capacity. Implementation of developed optimized support system, where only the position of stringer was modified, reduced the convergence of a gateroad by 25%. That value was proved by underground tests in the similar mining conditions.

Based on the above presented results it can be stated that numerical modelling codes supplemented by good quality input data, which allow to calibrate the models, are suitable for designing appropriate support schemes for given mining conditions.

Acknowledgement

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Parametric study of cable bolt performance under axial loading in medium strength synthetic rock

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Abstract

The Laboratory Short Encapsulation Pull-out Test (LSEPT) has been used to test the performance of cable bolt under axial loading ever since the 1970s. Two basic types of LSEPT are available in mining industry at present, one is the unconstrained test which allows the rotation of cable bolt during pull out and the other is constrained non-rotation test. A new pull out test facility and methodology which is able to prevent the free section of cable bolt from rotation during test was recently developed at UNSW Australia recently. Research is currently underway investigating the effect of various parameters on the cable bolt performance using the newly developed test facility. A series of pull out tests were conducted on the cable bolts embedded in the medium strength synthetic cylindrical rock with the strength of approximately 42 MPa. The parameters studied include borehole size and strength of grout. It was found that strong grout could have positive impact on the performance of cable bolt while the borehole size had negative effect on cable bolt support system.

1 Introduction

The axial bond strength of a bolt is usually determined by loading the fully grouted cable bolt along its axis. To date, many LSEPT facilities and methodologies have been developed based on the standard “split-pipe” test first reported by Fuller and Cox (1975). It is observed that different test facilities and methodologies might result in different load capacity of cable bolt.

Basically, there are two types of LSEPT widely used in mining industry, namely, unconstrained and non-rotating tests summarised by Hutchinson and Diederichs (1996). Bawden et al. (1992) compared the response of pull out tests when the rotation is allowed to that when it is prevented and they presented the test results in Figure 1. This reveals that the anti-rotation test facilities result in a better performance than the rotation ones. The almost plastic behaviour of cable bolt in rotation tests could be explained by the slip between the bolt and grout in the absence of dilation. However, the higher load capacity in non-rotation tests was resulted from additional friction component due to the mobilised geometry mismatch between cable bolt and grout.
Figure 1 Comparison of two types of pull out tests (Bawden et al. 1992)

1.1 Unconstrained pull out tests

In this approach, the cable bolt is encapsulated into the confining medium such as rock or steel pipe while the remaining section of the cable bolt is left free for gripping. Therefore, this test configuration enables the free section of cable bolt to rotate when it is pulled axially.

Hyett and Bawden (1996) performed 75 pull out tests using this type of test facility to investigate the effect of bulb frequency on the behaviour of grouted bulb cable bolt. They also studied the dependency of the performance of grouted bulb cable bolt on the radial stiffness of confining materials and embedment length. In the experiment, the cable bolts were embedded into both aluminium and steel pipes using a grout mix with a water to cement \((w:c)\) ratio of 0.4 with varying embedment lengths of 600, 900 and 1800 mm. Finally, they concluded that the bulb cable bolt has a higher bond strength and bond stiffness than the plain strand cable bolt. The test results also show that the stiffness of the grouted cable bolt is increased with the embedment length and radial stiffness of the confining medium. However, the stiffness would decrease with the bulb spacing.

Benmokrane et al. (1995) also used the unconstrained pull test approach to study the effect of grout properties on cable bolt performance. Instead of using pipes as the confining medium, a concrete cylinder with a diameter of 200 mm was used. It was concluded that the bond strength increases as the grout compressive strength and in their summary, the bond strength varies in proportion to the square root of grout compressive strength. In other words, the bond strength will increase as the \(w:c\) ratio decreases. However, they pointed out this mathematical relation might only be valid for their particular study.

This test configuration was also used by Chen and Mitri (2005). In the experiment, the effect of borehole size and \(w:c\) ratio on the performance of cable bolt were studied. The test results revealed no obvious relationship between borehole size and bond strength between bolt and grout. However, it was found that the grout strength does enhance the performance of cable bolt as it was clearly shown in Figure 2. This conclusion is consistent with that derived by Benmokrane et al. (1995).
Mosse-Robinson and Sharrock (2010) did a comprehensive study on cable bolt support system using unconstrained single embedment pull test facility as seen in Figure 3. The effect of grout strength, bulb frequency, borehole diameter and curing time on the performance of cable bolt were investigated. Steel pipes were used to provide the confining pressure and cementitious grout with two different w:c ratios, 0.35 and 0.45, were tested. Four different hole diameter were studied in the test including 42, 52, 81, 106 mm. After 37 pull tests, several conclusions were made. In terms of grout strength, it was found that higher w:c ratio might achieve varied range of bond strength while the lower w:c ratio grout always shows a more consistent and higher pull out strength. Again, it was observed that the grout strength can increase the load capacity and it is consistent with all the past research. Furthermore, the hole diameter does not have any discernible effect on the peak axial load as seen in Figure 4.

Figure 3  Test configuration (after Mosse-Robinson and Sharrock 2010)
1.2 Non-rotation pull out tests

This test approach is more complex and tends to increase the load capacity. As stated by Hutchinson and Diederichs (1996), the absence of rotation during pull out using this type of test facility forces the cable bolt to shear through the grout flutes and hence increases the bond strength. And it also more truly models the behaviour of a cable bolt under load in the field that occurs with bed or joint separation.

The anti-rotational test approach by Fuller and Cox (1975) was seen as the pioneering and foundation work in cable bolt axial test area. Their research was also the first in which the cable bolt instead of a single wire was tested. The test apparatus incorporated a single cable bolt grouted in two split steel pipes with a washer in between as seen in Figure 5. In the test, the effects of embedment length, the surface condition of the cable bolt and the surface shape of reinforcement member on the performance of cable bolt were studied. In terms of the surface geometry of cable bolt, they concluded that both the shape and condition of the cable bolt surface have critical effects on the load transfer between steel tendons and grout. The bond strength is expected to increase in the presence of indentation or rougher surface condition such as rust. Therefore, bulb is estimated to have the similar effect as indentation does by analogy.

Goris (1990) used a similar test arrangement as Fuller and Cox except use was made of barrel and wedge as anchor in the test apparatus. This study evaluated the effect of embedment length, the quantity of cable bolt grouted, w:c ratio and the curing temperature on the load transfer between steel tendon and grout. Regarding the w:c ratio, it was concluded that the lower ratio resulting in higher grout strength would enhance the load carrying capacity of cable bolt. This conclusion is consistent with the axial loading test of cable bolt by Stillborg (1984) and Reichert (1991).
However, the common shortcoming of this test apparatus is the pipe used for confinement of the cable bolt is significantly greater than rock. Considering this inconsistency, Thomas (2012) conducted a parametric study on 15 different types of cable bolts using a modified axial loading test facility. This set up used a diamond cored sandstone sample with a nominal UCS of 25 MPa and diameter of 142 mm as the confining material instead of steel pipes. In order to restrain the radial displacement, two thick split steel tubes were installed on the outer surface of the sandstone as seen in 6. The plain strand as well as bulbed and nutcaged cable bolts were tested in this study and hence it was concluded that the bulbed and nutcaged cable bolt typically produced higher load carrying capacity and onset residual strength. In terms of the effect of borehole diameter, the results showed that the peak load measured on the twin strand cable bolts was higher in the larger borehole while the opposite was observed for the plain strand cable bolt.

2 Methodology

The test facility used in this study is based on the design used by Thomas (2012) but with several important modifications. The modified design is described by Chen, Hagan and Saydam (2015) and the set up is shown in Figure 7. This facility is a non-rotation test apparatus and the confining material is the medium strength synthetic intact rock the details of which will be presented later.
Figure 6  Modified test facility with sandstone as confinement (Thomas 2012)

Figure 7  Schematic of the new LSEPT test facility (Chen et al. 2015)
The MW9 (indented) cable bolt manufactured by Megabolt Australia, was used in this parametric study as shown in Figure 8. It was expected that the bulb in the embedment section on MW9 (indented) would significantly increase the resistance between the bolt and grout during pull out compared with normal plain strand cable bolt.

![Figure 8 MW9 (indented)](image)

Ezy tube cylindrical cardboard moulds with length and internal diameter of 450 mm and 300 mm respectively as seen in Figure 9a were glued on board to form moulds to make synthetic rock test sample. The PVC pipe wrapped with the plastic tube with outer diameter of 5 mm and pitch of 20 mm was placed in the centre in the mould to make the borehole. The tube and piping was used to the borehole having consistent surface properties and a riffling effect of the boring machine driller on the borehole wall. The diameter of the PVC pipes was determined by considering the borehole requirement for MW9 cable bolts in the standard recommended diameter and an oversized borehole. The diameters of the two boreholes are shown in

Table 1. A cementitious mixture with the strength of 42 MPa was mix and delivered in one batch and poured into each mould to make test sample and Figure 9b shows the cast test samples after pouring. A mechanic vibrator was used to remove all the air bubbles in the cementitious mixture immediately after pouring.
The PVC pipes and Ezy tubes were removed 24 hours after casting and the test samples were then submerge in water for the remaining 28 days for curing as seen in Figure 10. The purpose of keeping all the samples in water is to maintain a constant moisture environment for test samples.

Figure 10  Test samples arranged in the water bath for curing

After curing, the lower 90 mm of borehole was backfilled with cementitious mixture to form a plug with the remaining 360 mm in the sample used as the embedment section. The cable bolt was then grouted into the borehole with grout of different water content and hence strength. It should be noted that the cable bolt was rotated in the grout during installation to maximize contact between bolt and grout. Figure 11 shows the prepared sample with installed MW9 (Indented) cable bolt.
A steel thick walled pipe termed the anchor tube was then placed over the remaining exposed section of cable bolt and secured with grout mix. The tube has a machined locking key slot on the outer surface whose function was to couple the test sample and anchor tube preventing rotation of the cable once load was applied as shown in Figure 12a. Figure 12b shows a thick section bearing plate linked the key to the test sample, its other purpose was to ensure even distribution of the load applied by the hydraulic cylinder over the top surface of the test sample. The grout used for steel tube installation has to be stronger than that used in borehole to restrict slippage of the cable bolt only occurred in the test sample. The assembled cable bolt and anchor tube was left to cure for 28 days before testing.

Prior to a test, a steel split cylinder was bolted together around the test sample (Figure 13a) with the annulus between the cylinder and sample filled with a cement mortar (Figure 13b). The steel cylinder provided the third and final element that coupled the anchor tube to test sample. By constraining the test sample, the cylinder also provided passive confinement to the sample. To ensure a consistent low level of confinement, the lock nuts were tightened to a constant 50 N·m torque.
A flowrate controlled hydraulic cylinder provided a fixed displacement rate for the cable bolt. A load cell and pressure transducer was used to record the force applied every second and a non-contact Micro Pulse measured the displacement every second. Figure 14 shows the assembled test arrangement prior to commencement of a pull out test. The displacement rate was fixed at 0.11 mm/s for all samples that resulted in test running time of 15 min.

3 Results and analysis

In this study, 12 samples were tested with three replications of each parameter. The testing conditions are shown in Table 2.

It was observed that failure only occurred by slip at the interface of cable bolt and grout as seen in Figure 15.


![Figure 15 Failure in all test samples occurred at the grout/cable bolt interface](image)

### 3.1 The effect of grout strength

The grout strength impact was studied for two borehole sizes, namely 42 and 52 mm. As shown in Table 3, the peak load for each condition was recorded as the load carrying capacity in kN and hence the average could be obtained. In terms of the tests with the borehole diameter of 42 mm, the average load carrying capacity for strong grout of 80 MPa UCS was determined to be 488 kN while weak grout with a UCS of 62 MPa resulted in a slightly lower peak load of 466 kN which is 22 kN or nearly 5% less. Similar results were observed in the tests with oversized borehole samples where the strong grout improved the load carrying capacity from 424 kN to 450 kN by 26 kN absolute increase equivalent to 5.8% relative increase. The differences in results for different scenarios can be seen in Figure 16. However, the difference of load carrying capacity for strong and weak grout tests could be seen small regardless of the borehole diameter. Overall, the grout with higher strength is more likely to increase the load carrying capacity and this conclusion is consistent with research by Benmokrane et al. (1995), Chen and Mitri (2005), Mosse-Robinson and Sharrock (2010), Goris (1990), Stillburg (1984) and Reichert (1991).

<p>| Table 3 Results of parametric study |
|-------------------------------|-----------------|-----------------|-----------------|</p>
<table>
<thead>
<tr>
<th>Borehole diameter (mm)</th>
<th>Grout Strength (MPa)</th>
<th>Load carrying capacity (kN)</th>
<th>Average (kN)</th>
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<tr>
<td>42</td>
<td>80</td>
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Parametric study of cable bolt performance under axial loading in medium strength synthetic rock

D. Li, P. C. Hagan, and S. Saydam

12 | Ground Support 2016, Luleå, Sweden

Figure 16  Load carrying capacity vs borehole sizes for different grout strength

3.2  The effect of borehole size

The effect of borehole size on the load carrying capacity can also be seen from Table 3. At grout strength of 80 MPa, it could be seen that the average peak load for a borehole of 42mm was 488 kN or 38 kN higher than that for 52 the mm borehole. This absolute difference is equivalent to an 8.4 % increase. Similarly, with the grout strength of 62 MPa, it could be observed that the smaller borehole size increased the load carrying capacity from 424 kN to 446 kN or 5.6 %. Figure 16 shows that the small sized borehole would enhance the load carrying capacity by between 4.5% and 5.8% for a 20 MPa increase in grout strength. Furthermore, this impact will be enhanced as the grout strength increases. It might be explained that the higher peak load is caused by higher confining pressure resulting from test sample rock in the small borehole sized sample. In other words, the volume of the confining material will have a positive impact on the load carrying capacity of cable bolt. However, this observation contradicted the conclusion by Chen and Mitri (2005) and Mosse-Robinson and Sharrock (2010) who found no relationship between the borehole size and load carrying capacity of cable bolt.

4  Conclusion

The new cable bolt pull out system designed in UNSW Australia is a modified version of the LSEPT test design. It is able to prevent the cable bolt from rotation during a pull lout test and compared to the earlier split steel pipe test method, uses material for confinement having properties closer to rock and hence better simulate the real underground environment. In this project, 12 test configurations studied the effect of grout strength and borehole size on the load carrying capacity of a cable bolt support system. In terms of the effect of grout strength, it was found that the grout with the higher strength enhances the performance of cable bolt system and this conclusion is consistent with the past research. In the case of increasing borehole diameter, however, it was found that load carrying capacity decreased. This finding conflicts with other research but this may be due to the positive effect of the volume of confining materials on the confining pressure to cable bolt. It is recommended that more samples with more borehole sizes should be tested to comprehensively investigate the effect of borehole size on the load carrying capacity of cable bolt system.
Acknowledgement

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References


Photogrammetric calculation of JRC for rock slope support design

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Abstract

For mining and civil engineering projects rock slope stability is an essential part of safety and financial considerations. While large-scale stability can be simulated using rock mass properties, at smaller scale local variations in rock properties become significant and failure purely along discontinuities is possible. Plane failures, wedging and rockfall are common occurrences at this scale. These are often prevented by bolting or shotcrete support. For temporary slopes such support measures can be costly, and an ability to simulate possible failures along discontinuity planes becomes useful for evaluation of support necessity. Conventional methods for evaluating rock mass properties require making assumptions for scale effect in upscaling for an effective slope scale. The scale effect is generally considered to be negative, while some studies show positive or no scale effect. Deriving parameters to elaborately represent mechanical properties of rock mass in a scale of a mining project would require conducting a large scale in-situ test. Representativeness of one test for a large region of interest is questionable. Photogrammetric recording of rock joint surfaces aims to automate the obtaining of rock joint roughness without the need for manual sampling and costly laboratory experiments. Measurements carried out manually using a profilometer are compared to results obtained with photogrammetry without human interaction. The results are compared to tilt table tests to obtain a ratio between the natural roughness to profiled roughness. This ratio may be interpreted as a measure of how well the joint surfaces match. The photogrammetric JRC values may be used to predict the strength of rock joints under low stress conditions, where dilatation is not prevented. Finally discussion is given concerning the applicability of the method and the need for future research.

1 Introduction

The idea of using photogrammetry to acquire JRC is not new. Recently, new numerical methods and development of customer level camera equipment have enabled the use of low-cost equipment to generate high resolution geometrical data. Nilsson et al. (2012) used off the shelf equipment and reached vertical accuracy of 0.5-1.0 mm compared to laser scanning. Uotinen et al. (2015) used a similar method and reached a point density of 16 pts/mm². Iakovlev et al. (2016) demonstrate how the digitally measured values compare to manually measured ones and attempted to convert the constant normal load test results to constant normal stiffness by removing the contribution of dilatation. In this paper, we continue the research to develop practical tools for rock slope support design. Our approach is suitable for plane failures, wedge failure and for rockfall. First we introduce the problem at hand, the photogrammetric approach used and the method to calculate the digital JRC. Next, the manual profilometer measurements are compared to photogrammetric results. Additionally the results are compared to tilt table tests. Finally, we discuss the results and give our recommendations on future research.

Stability of rock mass is an essential issue in many mining and civil engineering projects, as rock failures are a general safety issue. They are costly to clean up even when they do not happen to directly impact safety, and blocked passages incur losses due to the caused downtime. Often the rock is simulated as a single mass, only including jointing of the mass as a weakening parameter of the model. While this may work well...
for large-scale evaluations, it is generally accepted that at a small scale there are issues of joint stability and rockfall, shown by the tendency to attempt to prevent such events by the use of shotcrete and bolting. In some cases, especially those of temporary slopes and tunnels, using such methods of strengthening the rock can become costly and certain risks are accepted instead. It is however possible to make a visual overview of an exposed rock surface to estimate the need for additional investigations, i.e. simulations of failure risks. Based on such simulations, risks can be reduced by either forcibly removing the risky area e.g. using blasting, or by using appropriate support methods. This study focuses on finding methods for obtaining parameters and simulating such small-scale failure risks. (Iakovlev 2015)

Fractures or discontinuities in rock play a significant role on rock mass stability and this is taken into account in failure criteria as a parameter of how fractured the rock mass is, rather than behaviour of single discontinuities, as large-scale failures are almost invariably caused by a combined failure of the components of the blocky product of existing discontinuities. (Muralha et al. 2013, Bandis et al. 1981). This is, of course, an inevitable simplification, as it is nearly impossible to know the exact structure of discontinuities within a large rock mass. For blocky rock mass, the shear strength of single discontinuities may become critical.

Evaluation of the strength of a single discontinuity builds on Mohr-Coulomb relationship between the peak shear strength and the normal stress (1), which represents a case of a planar surface. This relationship has been expanded by Patton (1966) to represent the relationship for a rough surface (2) in a low normal stress with shear displacement occurring as sliding along inclined surfaces. At higher strength the asperities will break and the true relationship takes a different form. These frictional characteristics were first studied by Barton and Choubey (1977), and then expanded by Barton and Bandis (1990) to the commonly recognized Barton-Bandis criterion for rock joint strength and deformability (3).

\[
\begin{align*}
\sigma_s & = \tau_f + \tau_p \tan \phi_b \\
\sigma & = \sigma_n + \tau_p \\
\phi & = \phi_b + \phi_a
\end{align*}
\]

Where:
- \( \tau_f \) is the peak shear strength,
- \( \tau_p \) is the normal stress,
- \( \sigma \) is the cohesion,
- \( \phi_b \) is the angle of friction,
- \( \phi_a \) is the basic friction angle,
- \( \alpha \) is the angle of asperity,
- JRC is the joint roughness coefficient,
- JCS is the joint wall compressive strength and
- \( \phi_r \) is the residual friction angle.

The components for shear strength of discontinuities are illustrated in Figure 1. The base component is the basic frictional component, which represents a direct relationship between normal stress and shear strength based on frictional characteristics of the rock, weathered or not. On top of that any roughness, composed of geometrical variability of the joint and any asperities it has, creates extra shear resistance. It has however been shown that often the effect of the roughness component decreases as the joint length increases. This is partly due to the change of effective roughness with scale, and partly due to changes in
intact asperity strength. Thus, a rougher joint can be expected to be affected by scale more than a smoother one. On the other hand, behaviour of jointed rock mass cannot be directly deduced from joint behaviour studies alone. For instance, it has been shown that closely jointed rock mass with smaller blocks is likely to have higher peak shear strength than one with larger blocks. (Bandis et al. 1981)

Figure 1 The two shear strength components of basic friction and joint roughness, and their length scale effect according to Bandis et al. 1981. In general, longer joint samples lead to reduced shear strength. There is, however, no universal relationship for all joint types

Though it is recognised that mechanical behaviour of rock joints vary as a function of scale, the size and direction of this effect is not consistent across studies. Many studies show negative scale effect, which means decreasing strength with increasing joint size, consistent with the idea of shear strength consisting residual friction plus a scale-dependent roughness component. Other studies show positive or no scale effect. Summary of previous studies that examined the scale dependence can be found in Tatone & Grasselli 2013.

Overall the jury is still out on the defining factors of scale effect, however at least lower roughness can be assumed to diminish it, while correlation of roughness and undulation can determine the direction of the scale effect. For example, smooth joints with high undulation could be expected to have a positive scale effect, while rough planar joints would have a negative scale effect. Studies comparing well matching joints (Kutter & Otto 1990; Leal-Gomes 2003) suggest a positive scale effect for fresh well-matched joints, possibly due to higher effective undulation in matching joints, as mismatched joints have virtually no effective undulation.

Both natural and man-made slopes of various sizes are a common occurrence in many civil and mining engineering projects, e.g. road embankments and open pit mine slopes. Slope design includes consideration of economic, environmental and safety factors, of which slope stability focuses heavily on the safety aspect. However, consideration of economic risk means that slope design is also affected by economic factors, i.e. costs of slope failure compared to costs and probability of stability, especially in mining projects. In general, slope stability design consists of determining acceptable safety factors or probabilities for larger failures as well as rockfall, obtaining necessary geological and geotechnical information and simulating slopes using some method of analysis. A safety factor is defined as the ratio of maximum forces resisting failure to forces driving failure. For more temporary slopes, using failure probabilities may be appropriate, as clean-up and other failure costs are considerably smaller than for e.g. a long-term civil engineering project, where reasonable certainty of long-term stability is preferred. (Iakovlev 2015)
To simulate a slope the geotechnical properties of the slope material are necessary, as well as understanding of groundwater and climate conditions. Often literature values are used based on rock or soil type, as laboratory tests can be costly, time-consuming and inaccurate due to scale effects and other reasons. When past failure data is available, parameters can be back-calculated by simulating the failures. Geotechnical properties include material strength(s), weathering, and water permeability, as well as geological structure of the area, especially of discontinuity and weakness zones. Armed with the necessary parameters, which can be expressed as deterministic values or probabilistic distributions, a slope is simulated and analysed for failure possibilities. For failures beyond the height of several meters, simulation is done as rock mass analysis. For smaller failure possibilities, where failures might occur purely along single discontinuities, discontinuity stability analysis may be used.

Limit equilibrium analysis can be used when only a safety factor needs to be obtained, and it is computationally light, though it requires simplification of geometrical and geological data. Numerical methods, on the other hand, are used to more fully simulate the response of rock mass to various conditions, such as faults and groundwater conditions. Due to limits on computational resources, many discontinuity properties are input as weakening parameters of the rock mass, and only critical discontinuities and weakness zones are simulated as separate zones. However, it has been noted that slope failures advance through internal discontinuities and weakness zones of a rock mass, and lately work has been done to incorporate such a failure method into slope stability simulation. (Suikkanen 2014).

2 Methods

Eight rock joint surfaces were tested for evaluation of roughness characterization between traditional hand-measured approaches and photogrammetric roughness analysis in the rock mechanics laboratory of the department of civil engineering at Aalto University. The sample set consisted of three diorite samples and five glimmerite samples. The samples were collected by hand from post-blast and post-collapse sites by carefully removing loose surfaces with a hammer or by selecting visually undamaged joints from leftover rocks. The research extends studies conducted by Iakovlev et al. (2016), by expanding the sample pool with five additional samples and implementing an image processing routine for enhancing the accuracy of the photogrammetric modelling process.

The joint roughness parameters were obtained with joint tilt tests, by hand measuring with profilometer and by hand-measuring surface roughness amplitudes from the sample surfaces. The hand measured length normalized profilometer JRC reading was used as the benchmark variable. The basic friction angle ($\phi_b$) for the samples were determined by using a three-core tilt test. The joint compressive strength (JCS) of each sample was determined by using Schmidt Hammer tests. The JCS values did not differ significantly from UCS values determined in previous laboratory tests. The obtained parameters are provided in the results and discussion section. Finally, a photogrammetric procedure for roughness evaluation was conducted for inspection of applicability of photogrammetric calculation of JRC for estimating the friction angle of a joint surface.

2.1 Photogrammetry

The camera used for photographing of the samples was a Canon EOS 600D system camera with Canon EF 35mm f/2 IS USM objective lens, which has low optical distortion. The joint samples were photographed indoors in a consistent lighting environment with 4500 lx illuminance, measured at the center of the sample surface. ISO value was set to 100 to keep the noise in minimum. The shooting distance of 100 cm was selected to enable the sample surfaces to fit the images as whole. Aperture value of f16 was selected according to the shooting distance in order to fit the complete sample surfaces in the sharpness area of the photographs. Semi-automatic shooting mode was utilized for the exposure time to be selected automatically by camera enabling optimal exposure.

While photographing the whole sample surface to be photographed should fit in the picture. When optimal distance is being determined, it should be taken into account that the sample also fits the sharpness area,
which may be at an angle to the sample surface. The sharpness area is defined by the near distance of sharpness and the far distance of sharpness. The sharpness area is commonly referred to as depth of field (DOF). The principle of DOF is presented in Figure 2. The DOF can be calculated when the focal length, aperture value, subject distance and circle of confusion (CoC) are known. CoC for Canon 600D is 0.019 mm.

For determination of the total DOF, the near limit for sharpness is calculated with Equation (4) and far limit for sharpness with Equation (5). For these equations, a hyperfocal distance is needed, and it can be calculated by using Equation (6). (Greenleaf, 1950). Hyperfocal distance considered to be the nearest distance in which objects at infinity are acceptably sharp when lens is focused to this distance. In that case it means that all objects from infinity to one-half of hyperfocal distance are acceptably sharp.

\[
D = \frac{s}{f - c} \quad (4)
\]

\[
D = \frac{H}{s - f - c} \quad (5)
\]

\[
H = \frac{sq}{s + fc} \quad (6)
\]

Where:

- \( D \) stands for near distance of sharpness (mm),
- \( H \) is the hyperfocal distance (mm),
- \( s \) is the focus distance,
- \( f \) is lens’ focal length (mm),
- \( F \) is the f-number and
- \( c \) is the circle of confusion (mm)
Determination of the depth of field yields the required f-number for fitting the samples in the DOF area. It should be noted that the ideal focus point may be closer than the center of the sample surface. Especially at very close distances, the required f-value can be quite high (f16 or higher), which may introduce diffraction on the photos. While a general rule of thumb suggests the optimal f-value to be two steps above the lowest f-value of the lens, a more elaborate analysis of lens performance is suggested. The chosen lens is considered to have virtually no distortions (-0.2 % barrel distortion) to worry about, and it maintains very good performance in terms of diffraction till f11, and at f16 the performance is still considered to be acceptable with approximately 2000 MTF50 LW/PH (line widths per picture height). For comparison, the maximum resolution for this lens camera combination is 2600 MTF50 LW/PH at f4.

Lightning conditions remained unchanged during the photoshoot, as photographing was carried out indoors in a room without windows. Lightning was created by utilizing room’s 6 fluorescent lamps (3 on each side of the room), the nearest lamp at a distance of 2.2 m from the central point of the sample. Each lamp had two Philips TL-D 36W/830 fluorescent lights with length of 1.2 m. Also three fluorescent lamps were set above the samples in height of approximately 50 cm from the sample surface. These three lamps had each two pieces of Philips TL-D 58W/830 fluorescent lights, each with length of 1.5 m.

The imaging configuration is illustrated in Figure 3. The photoshoot was conducted by applying a camera stand for the camera. This configuration removes possible motion blur from the images, and enables imaging in low light conditions, as the possible movement of the camera during a shot is eliminated. The samples were placed on a rotational platform and rotated around the center point of the sample surface so that the angle of view differs 20 degrees between two consecutive photographs. This results in 18 images per one rotational round. The photography was conducted in three layers, corresponding to 37, 54 and 67 degrees from the horizontal plane defined by the sample surface. The photoshoot delivers 54 images from each sample. Remote control was applied for the photographing, so that no physical interaction with the camera is required during a rotational round. This assures that the camera position remains unchanged between shots. Changing from one layer to another on the other hand requires readjusting the camera position.

![Figure 3](image-url)

**Figure 3**  The imaging configuration viewed from side (on left) and from above (on right) with 54 camera locations in three rings at angles 37, 54 and 67 degrees containing 18 locations at 20 degree intervals as presented in Iakovlev et al. (2016)
2.2 Image processing

The produced RAW images were converted to JPEG file format by using Canon Digital Photo Professional software version 3.14.47. The resulting images were post-processed by an image processing routine to segment potential error sources out from the images. The primary concerns were to eliminate the unchanged background in every image, and to segment out other areas outside of the DOF region. The routine implements an iterative image segmentation by Protiere and Sapiro (2006) that utilizes geodesic computation by generalizing weights for the geodesic distance. This routine is a semi-automated segmentation that requires user to provided seed regions describing the areas to be split. As the surface samples rotate between images, the seed regions were selected to separate the background, the sample surface and rotational platform from the images. Different shooting angles require the areas to be defined separately, but as the samples are almost of the same size, one setting is sufficient for one shooting angle. All images from one shooting angle were processed with the same regional definition, and minor changes to the regions was required when changing to another shooting angle.

2.3 3D Model formation

The 3D point cloud formation was conducted with VisualSFM 0.5.25 software. The software utilizes SFM (structure from motion) technique that localizes features from 2D images and builds a 3D model reconstruction according to localized features (Ullman, 1979). Then images are matched with each other by SiftGPU algorithm. The SiftGPU implemented in VisualSFM is modified version of Lowe’s (1999) SIFT-algorithm, which converts the photographs to collection of local feature vectors to find and compare matching features between different photographs. SiftGPU distinguish special features, called DoG key points (Different of Gaussian), from the photographs. Found DoG key points are analyzed between photographs to find matching features. (Wu, 2007) The matching features are then combined to a sparse 3D point cloud by a multicore bundle adjustment routine (Wu et al. 2011).

The sparse reconstruction is then expanded to a dense reconstruction by PMVS/CMVS (Furukawa, 2010) routine. The CMVS (Clustering Views for Multi-view Stereo) creates clusters i.e. part models, which are then combined with PMVS (Patch-based Multi-view Stereo) function to one dense point cloud. The resulting point cloud is then saved as PLY (polygon file format). After point cloud construction, redundant parts are cropped off with Cloud Compare 2.6.0 software. Finally, the point cloud can be triangulated by applying a 2D-Delaunay triangulation (Delaunay, 1934) for the best fit plane of the surface, scaled to actual size with known dimensions, and saved in STL (Standard Tessellation Language).

2.4 Photogrammetric JRC calculation

The photogrammetric JRC calculation is conducted by following a procedure presented in Iakovlev et al. (2016). The reference coordinate system for the digital surface models is established by SVD (Singular Value Decomposition) routine, where the orthogonal base vectors are set as the new coordinate system, as illustrated in Figure 4a. After establishing the coordinate system, a sectioning plane is defined by taking a dot product of the base vector in the shearing direction and the plane normal. The sectioning routine is illustrated in Figure 4b. Then a search routine is applied to derive the 2D roughness profile in the shearing direction. The roughness profile is established by calculating the intersections of the shearing plane and the surface triangles that are hit by the plane from the corresponding line and plane equations.
The roughness characterization is performed by digital JRC calculation with slope length method (Tse & Cruden, 1979) by applying a normalization of sectioning plane by 0.5 mm sampling interval. The sampling interval was selected according to the sample window used for originally deriving the applied function. The normalization procedure is conducted by taking the mean value for height in a sampling interval, as this sampling resulted in the best match for studies conducted in Sirkiä (2015). The slope length method applies the root mean square (RMS) value from the local slopes of the profile with intervals between measured data points. The relationship with JRC and RMS presented as,

\[ Z = \text{RMS} \]

\[ \text{JRC} \]

where:
- \( Z \) stands for the RMS,
- is the height of the profile above reference line,
- the quantity of measures and
- the distance between measures.

3 Results

The sample surfaces were analysed for definition of a reference measure to be used in validation of digital JRC analysis. The JRC measures for the surfaces were evaluated by measuring three profiles for each sample, and recording the amplitude, profile length, and the Barton comb profile for each profile line. The profilometer measurements were compared against the roughness profiles corresponding to JRC values according to Barton and Choubey (1977). The amplitude measurements were converted to JRC values. Finally the derived JRC values were scaled to effective JRC measures with scale transformation proposed by Barton and Bandis (1982),

\[ (9) \]
and refer to 100 mm laboratory scale samples and and refer to sample size.

The JRC values derived by digital roughness characterization routine and by the traditional measures are presented in the Table 1. The effective JRC values are presented next to the derived parameters. Finally the average and the highest values for derived JRC parameters are presented for a comparison of effective roughness. A comparison of JRC values from different evaluation methods are presented in Figure 5, with manual profilometer measurements and digital predictions presented on the left side, and a comparison of JRC derived from tilt table testing with hand measured JRC presented on the right side.

Table 1 Results of JRC digital and hand measurements. 0 denotes raw measurement and N denotes normalized measurement using Equation 6

<table>
<thead>
<tr>
<th>Sample</th>
<th>Length (mm)</th>
<th>JRC(0) Highest</th>
<th>JRC(N) Highest</th>
<th>JRC(0) Mean</th>
<th>JRC(N) Mean</th>
<th>JRC(0) hand</th>
<th>JRC(N) hand</th>
</tr>
</thead>
<tbody>
<tr>
<td>Diorite-1</td>
<td>113.6</td>
<td>15.3</td>
<td>14.7</td>
<td>12.8</td>
<td>12.3</td>
<td>10-12</td>
<td>9.7-11.6</td>
</tr>
<tr>
<td>Diorite-2</td>
<td>112.4</td>
<td>16.0</td>
<td>15.4</td>
<td>14.4</td>
<td>13.9</td>
<td>14-16</td>
<td>13.5-15.4</td>
</tr>
<tr>
<td>Diorite-3</td>
<td>92.1</td>
<td>15.6</td>
<td>16.0</td>
<td>14.3</td>
<td>14.7</td>
<td>12-14</td>
<td>12.2-14.3</td>
</tr>
<tr>
<td>Glimmerite-1</td>
<td>111.5</td>
<td>9.5</td>
<td>9.3</td>
<td>8.1</td>
<td>8.0</td>
<td>10-12</td>
<td>9.8-11.7</td>
</tr>
<tr>
<td>Glimmerite-2</td>
<td>124.6</td>
<td>12.2</td>
<td>11.5</td>
<td>10.4</td>
<td>9.9</td>
<td>14-16</td>
<td>13.2-14.9</td>
</tr>
<tr>
<td>Glimmerite-3</td>
<td>80.2</td>
<td>14.1</td>
<td>15.0</td>
<td>13.0</td>
<td>13.7</td>
<td>4-6</td>
<td>4.1-6.2</td>
</tr>
<tr>
<td>Glimmerite-4</td>
<td>93.3</td>
<td>12.2</td>
<td>12.4</td>
<td>10.3</td>
<td>10.5</td>
<td>4-6</td>
<td>4.0-6.1</td>
</tr>
<tr>
<td>Glimmerite-5</td>
<td>81.3</td>
<td>19.6</td>
<td>21.3</td>
<td>18.5</td>
<td>20.0</td>
<td>16-18</td>
<td>17.1-19.4</td>
</tr>
</tbody>
</table>

Figure 5 Prediction-observation comparison for the digitally generated highest predictions (vertical Y-axis) and manual profilometer measurements (horizontal X-axis) with the horizontal error bars correspond to two JRC units (left), and measured-derived comparison of JRC with manual measurements (horizontal X-axis) and derived from tilt table testing (vertical Y-axis) (right)

Some characteristics are not fully covered by the roughness profiles. Iakovlev et al. (2016) presents that the diorite samples have clear undulation, steppedness or extreme asperities that a two-dimensional profile is unable to describe. Additionally, the glimmerite surfaces are more uniform, but the Glimmerite-3 and Glimmerite-4 samples seem to be part of a large undulation, and the sample Glimmerite-5 has large diagonal fibre structure that may contribute significantly to mechanical behaviour of the surface.
The digital roughness parameters are generally in the same order of magnitude than the traditional parameters, but overestimates hugely the roughness of samples Glimmerite-3 and Glimmerite-4. This overestimation seems to be generated from small scale variations, as the curve comparison presented in Figure 6 matches better with the traditional measurements. Excluding these samples as outliers, the overall performance of the digital JRC calculation seems to be following the nominal line in Figure 5 (left). The JRC values derived from tilt table testing are clearly lower than the manual measured values (Figure 5, right). Iakovlev et al. (2016) presents that this effect might be resulting from poor matedness (Figure 6) in the tilt testing. The tilt testing results are presented in more detail in Table 2. The table also lists an estimated correlation with the hand measured JRC values and the JRC values derived from the tilt table testing. This correlation is expressed as tilt JRC per profile JRC, and can be considered to represent the matedness of the tilt sample. Dividing the tilt test JRC with the profilometer JRC (with implied JMC = 100 %) gives an estimate of the Joint Matching Coefficient, which is a percentage of the matedness.

![Figure 6 Perfectly matching joint with high matedness (JMC = 100 %) on left, poorly matching joint with low matedness (JMC = 0 %) on right. Modified from Zhao (1997)](image)

<table>
<thead>
<tr>
<th>Sample</th>
<th>$\phi_b$ (°)</th>
<th>$\phi_r$ (°)</th>
<th>JRC (tilt)</th>
<th>JRC (profile)</th>
<th>JCS (MPa)</th>
<th>JMC (tilt/profile)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Diorite-1</td>
<td>31</td>
<td>31</td>
<td>6.0</td>
<td>12</td>
<td>216</td>
<td>0.50</td>
</tr>
<tr>
<td>Diorite-2</td>
<td>29</td>
<td>29</td>
<td>5.3</td>
<td>8</td>
<td>207</td>
<td>0.66</td>
</tr>
<tr>
<td>Diorite-3</td>
<td>31</td>
<td>29</td>
<td>6.1</td>
<td>10.5</td>
<td>194</td>
<td>0.58</td>
</tr>
<tr>
<td>Glimmerite-1</td>
<td>19</td>
<td>17</td>
<td>6.2</td>
<td>9.0</td>
<td>25</td>
<td>0.69</td>
</tr>
<tr>
<td>Glimmerite-3</td>
<td>19</td>
<td>17</td>
<td>4.2</td>
<td>5.0</td>
<td>25</td>
<td>0.84</td>
</tr>
<tr>
<td>Glimmerite-5</td>
<td>19</td>
<td>15</td>
<td>11.4</td>
<td>13.3</td>
<td>23</td>
<td>0.86</td>
</tr>
</tbody>
</table>
Additionally, the digital roughness profiles calculated with the photogrammetric routine are compared with JRC curves, digitalized from curves presented by Barton & Bandis (1977). The results of this comparison are presented in Figure 7. The original roughness curves were measured with a profilometer and drawn by hand. As discussed by Tatone & Grasselli (2013), the authors wish to remind that JRC began as a curve fitting parameter. Additionally the subjectivity of JRC determination by profile comparison should be recognized, and it should be noted that the definition would benefit from expertise and experience in analysis of rock surfaces. Matching the photogrammetrically derived digital profiles to original JRC curves provides lower JRC values than the photogrammetric JRC calculation with slope length method. These measures, while more subjective to the computerized calculation, seem to be matching more accurately the traditional parameters derived by profilometer measurements.

4 Discussion

In this study we used the camera on a stationary tripod and broad array of lights to emulate the evenness of cloudy day lighting. We could record the rock joint in continuous circles of camera locations at three different angles. These requirements cannot be met when performing in-situ photogrammetry in open-pit or underground mines. Technical requirements cannot be set for every foreseeable combination of conditions and equipment. However, a method to establish the acceptable limit should be possible to describe explicitly. Such a description should include how to determine the geometry for the photogrammetry, how to determine the resolution requirement, how many pictures are needed, how to
measure the required amount of light. The post-processing of the images differs very little between laboratory and in-situ photogrammetry.

Poturovic et al. (2015) carried out two series for replicas in constant normal load and constant normal stiffness conditions. They showed that the friction angle and dilatation are key factors that determine the shear strength of rock joints. For underground, the CNS condition is more realistic and dilatation is suppressed. Both CNS and CNL produce the same peak strength. It may be possible to numerically remove the dilatation to simulate CNS condition response from CNL test data. One approach could be to use the Patton’s (1966) interpretation of dilatation as a tangent between load and displacement. In CNS the suppressed dilatation is then translated into additional normal force.

To test the method towards larger scales, an affordable option would be to construct a large 2 m x 1 m tilt table and do blind predictions and then perform tilt tests. If these results are correct, the next step would be to construct a static load CNL shear test. This is the laboratory scale limit, but open pit mines provide opportunities for back-calculation of wedge failures. To overcome the effect the unknown variables (weathering, blasting induced damage, exfoliation, thermal effects e.g.), large quantities of failures should be analyzed simultaneously.

KTH Royal Institute of Technology and Aalto University have carried out small scale shear testing and scanned the surfaces using photogrammetry. These samples may be used as a benchmark for numerical simulations. If the numerical results can be calibrated to match the experiments, testable predictions can be given both for smaller samples (subsampling) and for larger samples (upsampling, e.g. large scale tilt table test). It would be beneficial to use several numerical codes to validate the codes to experimental results.

Finally, the numerical modelling of jointed rock mass should be further developed. This includes a more realistic fracture network with fracture set sequencing and termination. This enables the photogrammetry to record a fracture network intersecting the open surfaces and then generating the corresponding minimum energy extension to inside the rock mass. Ultimately, this results in a rigid synthetic rock mass. If particle models are added the model becomes full synthetic rock mass: crystalline material, fracture network and mechanical properties of rock joints. This may be considered as the long-term goal.

5 Conclusions

In this paper, we describe a method suitable for the digitization of rock joint surfaces. We describe a method to obtain a digital measure of joint roughness coefficient JRC. We have compared the results to hand measurements using a profilometer and using the amplitude method. Excluding two worst samples (Glimmerite-3 and -4), the digital JRC estimation works acceptably well and typically the roughness is digitally overestimated only by 1 JRC unit over the trend line. However, individual measurements rarely match the manually obtained values and the difference typically is 2-3 JRC units. The worst measurement (Glimmerite-3) has error of +10 units (+190 % overestimation). The best measurement (Diorite-2) just fits inside the upper limit of manual reading accuracy. We conclude that the method shows promise in ease of use and in reducing the subjectivity of JRC measurements. However, the deviation is currently too large and digital curve fitting appears to produce much better results. Further research is needed before the described method can be used in field measurements.

Acknowledgement

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References


Pipe Umbrella System—Dimensioning and Design

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H. Andersson, Huth & Wien Engineering AS, Norway

Abstract

Pipe Umbrella System is a temporary pre-support method for tunnels, suited for extremely poor geological conditions. The method is not commonly used in Norway, but is widely used internationally. There are many different techniques for installing Pipe Umbrella System, and the method is known by many names.

This article gives an extract of a master thesis focusing on the usage of Pipe Umbrella System in Norway, where different approaches for dimensioning and design was investigated. An empirical study has been carried out to search for a correlation between Q-value and type of Pipe Umbrella System. No correlation was found between Q-value and the choice of diameter of the steel pipes, distance between the steel pipes, or overlap between each umbrella arch. It was considered more advantageous to look at the ground conditions from a geotechnical standpoint. However, the cases in the study confirmed that Pipe Umbrella System is a method that successfully has been used for reinforcing the face of a tunnel in challenging geological conditions.

There are no generally accepted methods for designing and dimensioning Pipe Umbrella System. The method that is most used is the elastic foundation beam model. Therefore, an elastic foundation beam model was developed with the use of a construction program, and a parameter study was conducted with the model.

The master thesis was supervised by Professor Eivind Grøv and Dr. ing. Helen Andersson was co-supervisor.

1 Introduction

Construction of tunnels in unfavourable and poor geological conditions can give consequences like caving, fall out, inrush of water, or collapse of the tunnel. This can lead to delay of the project, economic loss, settlement at the ground surface and in worst case harm workers. Different methods are developed to excavate tunnels in challenging areas and Pipe Umbrella System (PUS) is one of these methods. Not many projects have used PUS in Norway, but during the recent years the method has gained popularity. PUS is constructed by drilling injectable steel pipes in an angle around the roof profile at the face of a tunnel, to keep the overburden from collapsing into the tunnel during construction.

Technical improvement of the method makes it possible to install the steel pipes in areas with hard rock and PUS is therefore an option when excavating tunnels through weakness zones and zones with small, or no rock overburden. There are many variants of PUS and the methodology has a great number of names, but little available descriptions of which ground conditions different methods and designs are suitable for. Several different methods for dimensioning and design of PUS are used, but there are no generally accepted criteria for dimensioning. The most used method is the elastic foundation beam model.

The PUS variant “Composite Pile Roofing” is used in several tunnel projects in Norway in the recent years, but officially the AT-Pipe Umbrella System used during the construction of a railroad tunnel in Holmestrand in 2013 is thought to be the first PUS in Norway. This is most likely because AT- Pipe Umbrella System has the most traditional design parameters considering type and diameter of steel pipes.

With this background the master thesis had main emphasis on suitability and dimensioning of PUS in Norwegian tunnelling.
2 General description of Pipe Umbrella System

PUS is a temporary pre-support method constructed by installing injectable steel pipes in the soil/rock mass, along the roof line of a tunnel, in front of the tunnel face (Figure 1). The method has similarities with the more common method known as spiling. The installed steel pipes are anchored to a lattice girder and grouted, before further excavation. Figure 2 shows installed steel pipes in a “Composite Pile Roof”.

PUS have traditionally been used in tunnels constructed in soil, but technical improvement of the equipment and the drill bits makes it possible to install in areas with hard rock. PUS can therefore be a suitable alternative in tunnels constructed through thick weakness zones (Sve et al., 2008). The main purpose of a PUS is to keep the overburden from collapsing into the tunnel and control settlement before permanent support is installed (Zhang et al., 2014).

Figure 1 Cross-sectional and longitudinal section of an installed PUS, with reference to design parameters (Oke et al., 2014)

The diameter on the steel pipes is commonly between 60 and 200 mm, with a wall thickness of 4-8 mm. The length of each pipe in an umbrella arch is usually between 12 to 15 meters, with an individual distance between each pipe of 0.4 and 0.6 meters. The umbrella arches overlap and the distance of overlapping is dependent on the ground conditions. The angle of the pipes from the horizontal is usually less than 15° (Volkmann and Schubert, 2007, Song et al., 2013).

The equipment used for installing the steel pipes can be conventional drilling rigs or custom made drilling rigs. For installation of the pipes it is possible to use “self-drilling” steel pipes equipped with disposable drill bits (one operation), or first drilling the hole before installing the pipes (two operations). When drilling in poor rock mass quality the hole can collapse and the installation of the pipe can be difficult. Under such conditions it can be favourable to use “self-drilling” pipes. The PUS can continuously be adapted to ground conditions encountered during construction (Volkmann and Schubert, 2007).

Figure 2 «Composite Pile Roofing» installed at Yxhugget (Photo: Roland Ekenberg)

After the steel pipes are installed, they are grouted. The grout fills the inside of the pipe, the space between the pipe and the surrounding ground, and potential fractures and spaces in the surrounding ground (Figure 3). The procedure of the grouting is different depending of the type of steel pipes used, where some steel pipes are perforated along the sides of the pipe, others are injected through the tip, and...
some can be injected during drilling. The goal of these methods is to get the grout distributed along the whole length of the steel pipes.

The grout have several positive effects: it reinforces the bending strength of the steel pipes, increases the load transfer and improves the stiffness in the surrounding ground (Volkmann and Schubert, 2008). By using deep threaded steel pipes the grip between the steel pipe and grout will also lead to a considerable shear load transfer (Ischebeck, 2010).

Figure 3  Cross-section of an injected TITAN Hollow bar (Ischebeck, 2010)

There are several different types of steel pipes available, with one-time drill bits for soil and rock (Sve et al., 2008). The two different types of steel pipes used in Norway is AT Ø113 from ALWAG and TITAN Hollow Bar 40/16 from Ischebeck (Andersson, 2015, Drageset, 2013). Table 1 gives an overview of the tunnelling projects where PUS has been installed as temporary pre-support in Norway.

Table 1  Norwegian projects where PUS are installed (Andersson, 2015, Drageset, 2013)

<table>
<thead>
<tr>
<th>Tunnelling Project</th>
<th>Name of PUS</th>
</tr>
</thead>
<tbody>
<tr>
<td>Nationalteateret (1997)</td>
<td>Composite Pile Roofing</td>
</tr>
<tr>
<td>Espatunnelen (2013)</td>
<td>Composite Pile Roofing</td>
</tr>
<tr>
<td>Holmestrand (2013)</td>
<td>AT-Pipe Umbrella System</td>
</tr>
</tbody>
</table>

Some of the most used terms on PUS are «Pipe Roof Umbrella», «Steel Pipe Umbrella System», «Umbrella Arch Method», «Pipe Forepole Umbrella», «Long-Span Steel Pipe Fore-Piling» and «Steel Pipe Canpy» (Volkmann and Schubert, 2007). According to Oke et al. (2014) there is no standardized terminology on the different varieties of PUS. Some of the most important design parameters are length of steel pipes, diameter of the steel pipes, installation angle, overlap between umbrella arches and individual distance between each steel pipe (Oke et al., 2014).

3 The function of Pipe Umbrella System as temporary pre-support

As previously described PUS are a temporary pre-support, before permanent support is established and is not counted as a part of the permanent support of a tunnel (Sve et al., 2008). Because of this some might say that it is more a method of excavation than a support method.
Immediately after installation the loading of the steel pipes will be close to zero, because they are not significantly affected by the prior excavation of the tunnel. But further excavation will activate loads from the overburden. The steel pipes will be exposed for the largest load in the unsupported zone close to the face. Behind the tunnel face the steel pipes are anchored to a lattice girder, which is part of the permanent support. In front of the face the steel pipes is supported of the soil/rock mass (Zhang et al., 2014). This way the steel pipes act as a supporting beam, illustrated in Figure 4. The strength of the PUS will also be influenced by ground conditions, the stiffness of the lattice girder and the shotcrete, which support the umbrella arch (Volkmann and Schubert, 2010).

The steel pipes only need to support the overburden for a relatively short period of time. When the excavation has passed the zone with the PUS and permanent support is installed, the PUS has no longer a function as support (Zhang et al., 2014).

4 Rock mass classification related to design of Pipe Umbrella System

An empirical study was carried out to look for correlation between rock mass classification and design of PUS. The Q-system is the most common method for rock mass classification in Norway and was therefore chosen for this study. Through literature studies it was found 20 PUS projects, these are listed in Table 2. Criteria for the projects were that they should be considered successful, with enough published data to calculate the Q-value and specifications for the design of the PUS. The projects cover PUS installed in different ground conditions as soil, weakness zones and missing rock overburden.

The Q-values estimated from the projects were plotted in Q-system support charts from Norwegian Geotechnical Institute (2013), and groups of design parameters was formed (Figure 5). The following plots were made:

- Overview of Q-values in all 20 projects.
- Groups sorted after diameter of steel pipes.
- Groups sorted after length of steel pipes.
- Groups sorted after overlap between umbrella arches.
Table 2 Selected projects with estimated Q-value and design parameters (Strømsvik, 2015)

<table>
<thead>
<tr>
<th>Tunnelling Project</th>
<th>Diameter Pipes [mm]</th>
<th>Wall Thickness [mm]</th>
<th>Length Pipes [m]</th>
<th>Overlap c/c pipes [m]</th>
<th>Estimated Q-value</th>
<th>Span/ESR</th>
</tr>
</thead>
<tbody>
<tr>
<td>Holmestrand</td>
<td>114</td>
<td>6.3</td>
<td>15</td>
<td>4</td>
<td>0.0250</td>
<td>11</td>
</tr>
<tr>
<td>Takakoa</td>
<td>60</td>
<td>-</td>
<td>12</td>
<td>3</td>
<td>0.0031</td>
<td>10</td>
</tr>
<tr>
<td>Fort Canning</td>
<td>114</td>
<td>6</td>
<td>12.5</td>
<td>3.3</td>
<td>0.0031</td>
<td>15</td>
</tr>
<tr>
<td>Dulles Corridor</td>
<td>150</td>
<td>-</td>
<td>18</td>
<td>4.6</td>
<td>0.0041</td>
<td>10</td>
</tr>
<tr>
<td>Elite Tunnel</td>
<td>168</td>
<td>7</td>
<td>12</td>
<td>3</td>
<td>0.0041</td>
<td>12</td>
</tr>
<tr>
<td>Yxhugget</td>
<td>36</td>
<td>10</td>
<td>15</td>
<td>12</td>
<td>0.0165</td>
<td>15</td>
</tr>
<tr>
<td>Erlangshan</td>
<td>102</td>
<td>10</td>
<td>30</td>
<td>-</td>
<td>0.0083</td>
<td>-</td>
</tr>
<tr>
<td>Istanbul Metro</td>
<td>114</td>
<td>6.3</td>
<td>9</td>
<td>3</td>
<td>0.0050</td>
<td>6.3</td>
</tr>
<tr>
<td>La Perosa</td>
<td>101</td>
<td>10</td>
<td>12</td>
<td>9</td>
<td>0.0075</td>
<td>13.5</td>
</tr>
<tr>
<td>Nathpa Jhakri</td>
<td>114</td>
<td>8.8</td>
<td>12</td>
<td>3.5</td>
<td>0.0038</td>
<td>12</td>
</tr>
<tr>
<td>Tujiangchong</td>
<td>108</td>
<td>6</td>
<td>19/40</td>
<td>0.42/0.4</td>
<td>0.0088</td>
<td>12</td>
</tr>
<tr>
<td>Caldecot 4th Bore</td>
<td>203</td>
<td>-</td>
<td>52</td>
<td>-</td>
<td>0.0135</td>
<td>15</td>
</tr>
<tr>
<td>Trojane</td>
<td>114</td>
<td>6.3</td>
<td>-</td>
<td>-</td>
<td>0.0175</td>
<td>11</td>
</tr>
<tr>
<td>Birgl</td>
<td>114</td>
<td>6.3</td>
<td>-</td>
<td>-</td>
<td>0.0050</td>
<td>12</td>
</tr>
<tr>
<td>Fiumelatte</td>
<td>84</td>
<td>4.5</td>
<td>12</td>
<td>4</td>
<td>0.0118</td>
<td>10</td>
</tr>
<tr>
<td>Delle Tanze</td>
<td>84</td>
<td>4.5</td>
<td>12</td>
<td>3</td>
<td>0.0546</td>
<td>7</td>
</tr>
<tr>
<td>Cernobbio</td>
<td>148</td>
<td>6</td>
<td>18</td>
<td>5</td>
<td>0.0144</td>
<td>11</td>
</tr>
<tr>
<td>Serre la Voute</td>
<td>140</td>
<td>10</td>
<td>14</td>
<td>4</td>
<td>0.0180</td>
<td>11</td>
</tr>
<tr>
<td>Pietragliata</td>
<td>101</td>
<td>10</td>
<td>18</td>
<td>4</td>
<td>0.0063</td>
<td>11</td>
</tr>
<tr>
<td>Serena</td>
<td>114</td>
<td>7</td>
<td>12</td>
<td>9</td>
<td>0.0046</td>
<td>13</td>
</tr>
</tbody>
</table>

The Q-values estimated in all the projects are within the exceptionally and extremely poor rock mass. Figure 5 a) shows that all except of one project has a Q-value between 0.003 and 0.03, which are support categories 7 and 8. Figure 5 b), c) and d) show no trends between Q-values and respectively diameter of steel pipes, length of steel pipes and overlap between each umbrella arch.

Available information regarding the ground conditions was in many cases sparse and it was challenging to estimate a reliable Q-value. Another factor affecting the estimation of the Q-value was that the Q-system is a classification system for rock mass, not soil, and in most of the cases the ground conditions consisted of soil. After the study it was seen as more suitable to classify the ground according to parameters related to soil.

During this study it was found that the most frequent complication when installing PUS is collapse of soil between the steel pipes. According to Furukawa et al. (2007) this complication can be solved by reducing the distance between the steel pipes and adjustments of the grout. The study confirmed the diversity of the method related to ground conditions and that PUS successfully has been used under several types of difficult ground conditions, as tunnelling through soil, weakness zones and missing rock overburden.
Figure 5  a) Rock support chart with plots of Q-values for all 20 projects, b) projects plotted with regard to pipe diameter, c) projects plotted with regard to pipe length, d) projects plotted with regard to overlap Strømsvik (2015)

5 Dimensioning by using elastic foundation beam model

Elastic foundation beam model is according to Zhang et al. (2014) the most common method for dimensioning PUS, and is performed by using finite element method (FEM). A variant of elastic foundation beam model was developed in the master thesis and is presented shortly in the following chapter.

5.1 Input parameters for elastic foundation beam model

The first step in the model is to find the input parameters. In this case the input parameters were:

1. Terzaghis Earth Pressure: pressure from the overburden.
2. Unstable zone: length of the zone where the earth pressure acts on the steel pipes.
3. Spring stiffness in the soil: the stiffness of the ground where the steel pipes are supported

When estimating the earth pressure generated from the overburden Terzaghis formula for failing horizontal support was found as the most appropriate. The concept is illustrated in Figure 6 and the formula given in Formula 1.
Figure 6 Illustration showing zone with soil movement caused of failing support between two solid bases (Terzaghi, 1943)

\[
\sigma = \frac{B}{K_0 \tan \phi} \left( 1 - \frac{q - K_0 \tan \phi \frac{B}{2}}{\sigma} \right) + q \frac{-K_0 \tan \phi \frac{B}{2}}{\sigma}
\]  

(1)

Where:
- \( \sigma \) = earth pressure
- \( \gamma \) = weight density of soil
- \( c \) = cohesion soil
- \( B \) = half the length of unstable zone
- \( K_0 \) = coefficient of earth pressure
- \( \phi \) = friction angle in soil
- \( q \) = potential load on the ground surface

Formula 2 is derived to calculate the length of the unstable zone. The basis for this formula is that Terzaghi (1943) found that average angle for potential sliding plane is \( 45^\circ + (\phi/2) \).

\[
2B = 2L_e + H_L \tan \left( 45^\circ - \frac{\phi}{2} \right)
\]  

(2)

Where:
- \( B \) = half the length of unstable zone
- \( L_e \) = length of one excavation round.
- \( H_L \) = depth of soil at the face of the tunnel.
- \( \phi \) = friction angle in soil

Figure 7 illustrates the concept of the potential sliding plane at a tunnel face and the unstable zone. The unstable zone is comprised by the last excavated round with permanent support which is shotcrete and lattice girder (1), the last round of excavation (2) and the zone that potentially can glide, or collapse into the tunnel (3). The reason for adding the last round of permanent support is because it takes some time before the shotcrete attain full strength.
To find the steel pipe in an umbrella arch which is exposed to the highest loading a calculation of normal force per running meter of each steel pipe around the profile at the face was carried out. The reason for this is to see the effect of the increasing overburden along the tunnel profile. The results, shown in Figure 8, indicate that the steel pipe at the top of the profile is experiencing the highest loading. The input parameters for this calculation are from the project Yxhugget in Stockholm and are presented in Table 3. These variables also represent the input parameters used to perform the elastic foundation beam model presented in this article.
Table 3  Input variables from Yxhugget in Stockholm (Andersson et al., 2011, Borchardt, 2006)

<table>
<thead>
<tr>
<th>Variable</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Friction angle soil, ( \phi )</td>
<td>35°</td>
</tr>
<tr>
<td>Weight density of soil, ( \gamma )</td>
<td>20 kN/m³</td>
</tr>
<tr>
<td>Cohesion soil, ( c )</td>
<td>10 kPa</td>
</tr>
<tr>
<td>Young’s modulus soil, ( E )</td>
<td>20 MPa</td>
</tr>
<tr>
<td>Poissons ratio soil, ( v )</td>
<td>0.4</td>
</tr>
<tr>
<td>Coefficient of earth pressure, ( K_0 )</td>
<td>0.5</td>
</tr>
<tr>
<td>Potential load on the ground surface, ( q )</td>
<td>15 kPa</td>
</tr>
<tr>
<td>Overburden, ( h )</td>
<td>16 metres</td>
</tr>
<tr>
<td>Length of steel pipes, ( L )</td>
<td>15 metres</td>
</tr>
<tr>
<td>Installation angle of the steel pipes, ( \theta )</td>
<td>8°</td>
</tr>
<tr>
<td>Depth of soil at the tunnel face, ( H )</td>
<td>1.7 metres</td>
</tr>
<tr>
<td>Length of one excavation round, ( L_e )</td>
<td>3.0 metres</td>
</tr>
<tr>
<td>Individual distance between each steel pipe, ( S )</td>
<td>0.4 metres</td>
</tr>
</tbody>
</table>

5.2  FEM analysis of elastic foundation beam model

Different types of software are available for solving construction problems based on FEM. In this master thesis it was chosen to use the software Frame Analysis Program (FAP), which is developed by students at NTNU.

The steel pipe that was estimated to experience the highest loading was designed in the drawing board in FAP. Input parameters for this task were length of steel pipe, installation angle, length of instable zone and spring stiffness of the ground. Figure 9 shows how this appeared in FAP. The boundary condition corresponding to the lattice girder is chosen to be fixed in this case, but it might be more correct to also use a spring where the stiffness of the lattice girder is assigned.

Figure 9  Elastic foundation beam model constructed in FAP (Strømsvik, 2015)

After the model was constructed an analysis with loading from the earth pressure was performed. The results display the calculated maximum moment and the maximum shear force the steel pipe is exposed to under such conditions, shown in Figure 10.

Figure 10  Calculated maximum moment and maximum shear force in FAP (Strømsvik, 2015)
5.3 Parameter analysis using elastic foundation beam model

It was conducted three parameter studies using the elastic foundation beam model:

1. Increasing depth of soil at the tunnel face from the roof towards the floor, 1-10 metres.
2. Increasing length of one excavation round, 0.5-3 metres.
3. Increasing individual distance between steel pipes, 0.2-0.7 metres.

![Graphs](image)

Figure 11 a) development of earth pressure, moment and shear force with increasing depth of soil at the tunnel face, b) increase in unstable zone with increasing depth of soil at the tunnel face, c) development of earth pressure, moment and shear force with increase in excavation steps, d) development of earth pressure, moment and shear force with increasing distance between each steel pipe, modified after Strømsvik (2015)

The results from the three parameter studies (Figure 11) indicate that in relation to loading of the steel pipes the individual distance between each of them is not as important as the depth of soil at the face of the tunnel and the length of the excavation steps. In this context it is important to add that the individual distance between each steel pipe is of great importance for upholding the arching effect in the soil, reducing the risk for soil collapsing between the steel pipes. This effect is described by Eckl (2012) and is illustrated in Figure 12.
5.4 Summary on elastic foundation beam model

It was concluded that the elastic foundation beam model can be a useful tool for deciding some of the most important design parameters for dimensioning PUS. By using relatively simple input data it is possible to make estimations related to the loading of the steel pipes and optimise the distance between each umbrella arch, in regard to the depth of the soil at the tunnel face. Individual distance between the steel pipes should be estimated by using different approaches, but is still an important input parameter in the elastic foundation beam model.

The model is conservative in the way that the length of the unstable zone includes the last round of permanent support and the soil over the potential gliding surface. The model does not include the effect of overlapping umbrella arches. Nor does the model take into account the strengthening effects of the grouting of the steel pipes, since it cannot be guaranteed that the grout enclose the whole length of the steel pipes.

6 Conclusion

In the empirical study it was not found any correlation between estimated Q-value and design parameters such as diameter of the steel pipes, distance between the steel pipes or overlap between umbrella arches. It was considered more advantageous to look at the ground conditions from a geotechnical standpoint. However, the case studies confirmed that Pipe Umbrella System successfully has been used for temporary pre-support of tunnels in challenging geological conditions like soil, weakness zones and missing rock overburden. During the study it was found that the most frequent complication when installing Pipe Umbrella System is collapse of soil between the steel pipes.

The elastic foundation beam model developed in this master thesis is considered useful for its purpose. The calculation model can be a good tool for dimensioning some of the most important parameters at a preliminary stage for the design of a Pipe Umbrella System, but the model lacks testing with data from more projects.
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Production-blast-induced crosscut performance: a comparison of three high-deformation bolt types

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Abstract

In 2011, a test was begun in a crosscut on the 1022 level of the Malmberget iron ore mine’s Norra Alliansen orebody. The goal of this test was to compare the performance of three rockbolt support regimes with the long-term performance of the crosscut. They were installed in the most problematic geology found in the mine, a weak biotite schist located primarily along the footwall contact. The three regimes included Kiruna bolts, D-bolts, and Swellex bolts, each instrumented with multi-point extensometers and strain-gaged bolts in an effort to capture the response of the bolts and the rock to both time and mine-induced stress changes. Differences were observed between the regimes in full deformation and strain profiles indicating variations in rockbolt response and performance as well as stress-induced patterns occurring in the walls, shoulders and back of each bolting profile.

1 Introduction

The monitoring associated with this report has been ongoing since mid-2011. It follows other work done in the Malmberget mine with the goals of better understanding the interaction between support and the rock mass, and identifying the need and timing for secondary support (Sundström, 2010; Nordlund, 2013). The current effort attempted to differentiate between the performances of various support types within the most challenging geology found in the mine, a weak biotite schist commonly found along the footwall contact zone. This schist has been known to cause difficulty for the mine, and can contribute substantially to the degradation of entries and crosscuts.

To help ensure stability and safety in this weak zone, a comparative bolting study is underway in a crosscut installed with multiple bolt types in the biotite. This is a simple and common method of determining bolt performance in the laboratory or in situ (Signer and Lewis, 1998; Grasselli, Kharchafi and Egger, 1999; Thompson and Villaescusa, 2014). Entry deformation and entry convergence have been monitored through the use of extensometers over a period of approximately four years. This can help in understanding deformation events and can lead to improved safety and lower costs for the mine (Bawden, Dennison and Lausch, 2000). The bonding and separation between shotcrete and rock is also important in ensuring stability (Malmgren, 2005; Malmgren and Nordlund, 2008). These have been monitored as well.

The following details the reaction of the rockmass and bolts to mine-induced stresses caused by production blasting around the study area. Only blasting occurring on the levels above the test has been considered as it has been shown that blasting on the same mine level as the instrumentation has little effect unless the blasting is relatively close (28 to 50 m) (Sundström, 2010; Jones, 2016).

2 Methods

Instruments were installed in a crosscut on Level 1022. A plan view of the area is shown in Figure 1, and the location within the level is shown in Figure 2. This geology is known to develop serious ground control problems characterized by large deformation and instability. The test area was divided into three adjacent test sections with Section 1 closest to the footwall. Each included five blast rings, labelled R1 through R5 in Figure 2. These rings will be blasted in the future when ore is removed from the crosscut itself, but are not otherwise used as part of this study.
Bolts installed in the test sections included NMX (Kiruna) bolts, D-bolts, and Swellex bolts. Sentinel integrity-monitoring rebar bolts with the same dimensions as the Kiruna bolts were also installed but were not used in this analysis. All were placed in a 1 x 1-m pattern and combined with 150-mm-thick shotcrete reinforced with Dramix 65/35 (40 kg/m³ of shotcrete), 35-mm long steel fibers with a length-to-diameter aspect ratio of 0.65. One fan of bolts in Section 1 and three fans in Section 3 were instrumented with strain gages, details of which can be found in Table 1 along with dimensional details of the bolts.

In each test section an extensometer and convergence profile was prepared by installing multipoint borehole extensometers (MPBXs) as shown in Figure 3. Anchors were installed at 1, 2, 3, 4, 5 and 7 m depth in the walls, and 1, 2, 3, 5, 7, and 10 m depth in the shoulders and roof. The distance between the face and the instruments was around 10 m at the time of installation. Convergence points were anchored from rock-to-rock and from shotcrete-to-shotcrete.

![Figure 1](image1.png)

**Figure 1** “Plan" view of instrumented area showing roof and walls

![Figure 2](image2.png)

**Figure 2** Map of level 1022 with the instrumented area circled in red

![Figure 3](image3.png)

**Figure 3** Elevation view of one extensometer profile, showing shotcrete liner, MPBX anchors, and convergence measurements. Opening dimensions are also present

<table>
<thead>
<tr>
<th>Section</th>
<th>Bolt-type</th>
<th>Length (m)</th>
<th>Diameter (mm)</th>
<th>Strain gages (m from threads)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Kiruna</td>
<td>3.05</td>
<td>20</td>
<td>0.200, 0.850, 1.525, 2.185, 2.850</td>
</tr>
<tr>
<td>2</td>
<td>Swellex (Mn 24)</td>
<td>3.05</td>
<td>36 (initial)</td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>D-bolts</td>
<td>3.00</td>
<td>20</td>
<td>0.280, 0.910, 1.780, 2.460</td>
</tr>
</tbody>
</table>

Table 1  Bolt details and instruments installed in each test section
The distance between each instrument and each production blast that occurred throughout the mine during the monitored time period was calculated from coordinates taken from 3D mine maps. Straight-line distances were calculated between the head-locations of each instrument and a point on the floor in the center of the crosscut, directly beneath each blast ring. Blasts occurred throughout the orebody on multiple levels. Greater detail about the method can be found in literature (Jones, 2016).

The dates of data collection did not correspond with the dates of blasting, and multiple blasts from different distances occurred on the same day. To allow direct comparison, derived units were used to determine the relationships between blast distance and instrument readings. Readings from the SMART bolts (Stretch Measurement to Assess Reinforcement Tension) were collected approximately once every 24.7 days over the course of the study period. Production blasts averaged once every 2.3 days over the same study period. Since there are also times when either the gap between blasts or instrument readings was significantly greater than these averages, average-weekly blast distance values were compared with calculated average-weekly deformation or deformation rate.

3 Results

Entry deformation

MPBX data was analysed with respect to production blast distance. Results were compared both from instrument position to instrument position (right or left, wall, shoulder or roof), and from test section to test section. The MPBX behavior in the different installation positions tends to have similar patterns and magnitudes regardless of the test section. Clear differences can also be seen between sections.

Deformation rate is largely a function of the distance between instruments and production blasting activities, as these are the primary drivers of mining-induced stress change. Time-dependent deformation is also an influence, though the deformation rate is generally lower. This deformation can be seen during periods where blasting is occurring at a great distance, or during long breaks in blasting. The time-dependent deformation is driven by in-situ stresses and previously-developed mining-induced stresses and has a relatively stable deformation rate across the entire monitoring period.

Deformation magnitude is a function of time and rock quality. Rock quality was similar over the small testing area. The deformation records include a number of intervals where blasting did not take place on the level above. In these spans the only influencing factor was time. Over these spans deformation occurred relatively similarly, with the rock surface moving around 13 μm per day in all test sections on average.

Left wall

Deformation magnitude was largest on the left wall of each instrument profile (Figure 4). Left wall deformation tends to have a smooth trend exhibiting a steady deformation rate for each anchor. Deformation decreases as the anchors progress deeper into the rock in each section. Differences in deformation rate are also consistent between anchors in each section: the 0-m anchor deforms the most, 14 to 22 mm, while the 1-m anchor converges slightly less. The 3, 4, 5 and 7-m anchors converge decreasingly less going deeper, with the 3-m anchor magnitude approximately one-fifth of the 0-m anchor amount. The 2-m anchor response varies between sections.

These trends indicate that the wall on the left side of the crosscut (looking towards the face) is deforming together as a unit. The confinement of the rock itself controls a great deal of the movement along the imbedded length of the instrument, and the bolts are holding the rock together. Compared to other instrumented positions in the drift profile, the magnitude is influenced less by blasting-induced stresses. In other positions there is a tendency for blasting to create sharply increased deformation rates, depending on the proximity of the deformation.
Deformation magnitude in the left shoulder locations was 5-10 times less than that in the left wall, and the pattern of deformation over time is less smooth (Figure 5.) Deformation rates tend to gradually increase over time along the entire length of the instrument, though blasting still creates sharp rate increases.

Inspection reveals that the location is highly-influenced by blasting. Production blasting occurring on the level above produced noticeable increases in deformation rate, especially after it had halted and then resumed. Every time the average-weekly blasting distance was less than 75 - 85 m there was a much more significant increase in deformation rate than with further blasting.
**Roof**

The roof deforms little compared to the left locations (Figure 6.) During the first 475 days it tends towards a slight extension, changing to gradually increasing convergence until the end of monitoring. This indicates that the stresses passing the roof are likely acting more horizontally, rather than pushing directly downward. Only the closest of the average-weekly blasting distances caused significant increases in deformation rate, generally at around 50 m.

![Figure 6 Typical roof deformation pattern](image)

**Right shoulder**

The right shoulder experienced the lowest overall magnitude of deformation of any of the five positions (Figure 7.) During the first 150-300 days there was a small extension (less than -2.0 mm), and in Section 1 this didn’t stabilize until around day 600. The extension tended to occur while the blasting was approaching, yet was still quite far away (greater than 200 m.) Following this period the deformation changed to convergence, though it did not always converge enough to overcome the negative deformation of the contraction. At the end of the monitored period, the overall deformation ranged from -1.49 mm in section 1 (increased from a minimum of -1.51 mm), to 0.57 mm (increased from a minimum of -0.63 mm).

![Figure 7 Typical right shoulder deformation pattern](image)
The shift in deformation tendency from extension to convergence occurred roughly when the blasting on the instrumentation level shifted to include blasting in the smaller, western section where the instruments were located.

**Right Wall**

Right wall and left wall instruments deformed very similarly with respect to deformation rate, but the right wall exhibited magnitudes around half of that found in the left wall. Nearby blasting caused convergence when the average-weekly distance was less than 75 to 85 m (Figure 8.)

![Figure 8 Typical right wall deformation pattern](image)

There is clearly a relationship between deformation rate and average-weekly blasting distance. The “impact distance” varies from instrument location to instrument location within each profile (Table 2). This impact distance is the greatest distance at which production blasts created jumps in the deformation rate of the data. As such, it can be considered a measure of sensitivity to blast-induced deformation where a greater impact distance relates to greater blast-sensitivity. Regardless of the instrument location, this dataset showed that when the average-weekly blast distance was greater than 200 m, mining-induced stress changes were indistinguishable from the virgin stresses. In these cases, time-dependent deformation is the driving force, giving the instruments steady deformation rates.

<table>
<thead>
<tr>
<th>Position</th>
<th>Movement range (mm)</th>
<th>Approximate impact distance</th>
</tr>
</thead>
<tbody>
<tr>
<td>Left wall</td>
<td>17.0-21.0</td>
<td>75-85 m</td>
</tr>
<tr>
<td>Left shoulder</td>
<td>2.2-5.5</td>
<td>75-85 m</td>
</tr>
<tr>
<td>Roof</td>
<td>1.98-3.4</td>
<td>50 m</td>
</tr>
<tr>
<td>Right shoulder</td>
<td>-0.63-0.57</td>
<td>175 m</td>
</tr>
<tr>
<td>Right wall</td>
<td>5.2-10.5</td>
<td>75-85 m</td>
</tr>
</tbody>
</table>

**Bolt strain**

Close attention must be paid to quality control while installing strain gages on rock bolts. In most cases matching slots must be cut along both sides of the length of the bolt. Having accurate and consistent
depths in these slots is important for ensuring high-quality strain data. Slight changes in the slot-depth change the moment of inertia of the bolt, which can give significant error at the microstrain level. This factor could have played a role in the quality of the data from the two bolts. The uneven surface associated with the rebar Kiruna bolts makes it more difficult to precisely cut the necessary slots, resulting in less-accurate results. The D-bar has a smooth-bar construction, and in this case the gages were glued directly to the surface rather than in a cut groove, likely improving gage installation.

The same rebar surface texture alters the way strain is distributed along the bolts. The D-bolt is specifically designed with the goal of allowing the sections between anchors to slide freely within the cement anchoring it in place. The Kiruna bolt is point anchored at its terminus, but is also fully-encapsulated. Unlike the D-bolt, the texture acts as its own anchor along the bolt, delaying the distribution of stresses along its length. This makes the Kiruna bolt more prone to point loading at a specific location if a joint or fracture develops in the rock. The stresses must overcome the anchoring strength of the steel rebar ridges within the cement. The resulting strain-gage measurements exhibit a jagged tendency, reflecting these construction-related factors.

**Kiruna bolts**

Axial bolt loads for both bolt types were calculated by averaging the results of the opposing gages in a given location (McHugh and Signer, 1999). All gages were configured in pairs and if one of them failed or returned faulty data, the result of a single gage was used given that the result was logical, otherwise, it was omitted.

![Kiruna bolt strain performance, left wall, section 1](image)

The bolt installed in the left shoulder of Section 1, ring 3, reached its maximum recorded value only 100 days after installation (Figure 9.) This is erroneous, as verified by the nearby extensometer. Additionally, the 0.86 m gage-pair had one malfunctioning gage that caused a negative offset error in the data. Otherwise, the gages appear to behave similarly, as would be expected given the extensometer results.

The highest load experienced was 106 kN located 0.200 m from the bolt threads. At the same time the nearby extensometer indicates that the bolt experienced 2.01 mm of extension between the 0.00 and 1.00-m anchors, resulting in a strain of 0.2%. This equates to a derived load of approximately 60 kN, a difference of 46 kN. This illustrates the impact of point loading due to the design of the Kiruna bolts, and makes interpretation of their actual strain readings difficult. Since the Kiruna bolt has a typical yield strength of 165 kN, and can withstand up to 55 mm of strain given a static load, it is likely that the bolt is still well within its elastic support capacity.
Though of poor quality, the results do indicate that the Kiruna bolt experiences increases in loading due to blasting. The distance data displayed on the extensometer graphs show that the blast distance was 100 m or less on certain days. Many of these events can be correlated with increases in the bolt loading indicated in Figure 9. For clarity they are outlined in Table 3. While not all gages increased in load every time the average blasting distance was less than 100 m, the bolt load as a whole did increase. The bolts are experiencing load increases from nearby blasting, and 100 m appears to be a good threshold limit or impact distance for load increase.

Table 3 Correlation between average-weekly blast distances less than 100 m (Figure 4-Figure 8) and bolt load behavior (Figure 9)

<table>
<thead>
<tr>
<th>Spike occurrence (day range)</th>
<th>Avg. Weekly Distance (m)</th>
<th>Load behavior</th>
</tr>
</thead>
<tbody>
<tr>
<td>494-500</td>
<td>77.6</td>
<td>0.86-m gage increases 19 kN; 1.53-m gage increases 5 kN; Other gages decrease 2-3 kN</td>
</tr>
<tr>
<td>641-647</td>
<td>73.9</td>
<td>No bolt reading</td>
</tr>
<tr>
<td>662-668</td>
<td>89.7</td>
<td>All gages increase from 10 to 60 kN</td>
</tr>
<tr>
<td>683-710</td>
<td>61.6</td>
<td>3 gages increase 3 to 27 kN, 0.86-m gage decreases 16 kN</td>
</tr>
<tr>
<td>902-906</td>
<td>43.4</td>
<td>0.2-m gage, no change; 0.86-m and 1.53-m gages increase 21 and 43 kN; 2.19-m gage decreases 23 kN</td>
</tr>
</tbody>
</table>

The other two Kiruna bolts had worse quality data than that seen in the left wall. The roof data had the same pattern of bolt compression and extension seen in the extensometer results, but was poor quality. The right-wall results indicated bolt compression, while the extensometer only lengthened.

**D-bolts**

The data recorded by the D-bolts was much higher-quality than that provided by the Kiruna bolts, due to the smooth-shaft bolt construction. An example from the left wall of section 3 is in Figure 10.

![Figure 10 D-bolt strain performance, left wall, section 3](image)

The D-bolt strain results matched the deformation results recorded by the extensometers very well with regard to both magnitude and location of strain readings along the bolt. In most cases, an impact distance could be determined for the bolts, as with the MPBXs. Below these distances, the instrumented bolts were more likely to show signs of strain-rate change due to blast proximity. The roof had an impact distance of 50-m with both extensometer and strain gage. Impact distance for the D-bolt strain gage installations can be found in Table 4.
Table 4 Maximum load results from instrumented D-bolts

<table>
<thead>
<tr>
<th>Position</th>
<th>Maximum load (kN)</th>
<th>Impact Distance (m)</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>Left wall</td>
<td>120</td>
<td>100</td>
<td>Good match with MPBX data</td>
</tr>
<tr>
<td>Left shoulder</td>
<td>N/A</td>
<td>N/A</td>
<td>Data error: no zero point determined</td>
</tr>
<tr>
<td>Roof</td>
<td>12</td>
<td>50</td>
<td>Good match with MPBX data</td>
</tr>
<tr>
<td>Right shoulder</td>
<td>15</td>
<td>100</td>
<td>Good match with MPBX data</td>
</tr>
<tr>
<td>Right wall</td>
<td>82</td>
<td>None</td>
<td>Good match with MPBX data</td>
</tr>
</tbody>
</table>

Tape extensometer convergence

The results from the tape extensometer are displayed in Error! Reference source not found.. The individual data points are shown on each of the lines. Due to the low temporal resolution during the middle part of the measurement record, it is impossible to see the effect each time the average blast distance fell below 100 m.

The convergence measurements help to identify the performance of the rock bolts and shotcrete with respect to deformation. The two convergence lines from Section 1 are nearly identical throughout the entire measurement range. At the end of the monitoring period, 1150 days after installation, the shotcrete has only experienced 0.75 mm more convergence than the rock in Section 1. This is small compared with the 5.62 mm in Section 2 and 3.3 mm experienced in Section 3. Given these results, it appears that the Kiruna bolts performed the best at preventing separation between the rock and shotcrete. The D-bolts had a middle performance, while the Swellex bolts allowed the greatest shotcrete separation.

4 Discussion

The instruments in this study have been installed in a manner that should allow for some determination to be made about their suitability for use in the difficult conditions found underground in Malmberget. Of particular interest is their functionality when installed in the weak biotite schist found along the footwall of the Norra Alliansen ore body. By looking at their performance it was also hoped that a difference could be
Production-blast-induced crosscut performance: a comparison of three high-deformation bolt types

T.H. Jones

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determined between Swellex bolts, D-bolts, and Kiruna bolts, and that a better understanding could be developed regarding the deformation of the entry and how the bolts interacted with it.

A great deal of correlation can be found between the different data sources used in this project. Of most importance, the results from the extensometers are verified by those returned by the strain-gage instrumented rock bolts. The different installations of extensometers provided similar responses regardless of which row they were installed in, meaning that the test site served well as a location for comparing bolt performance.

Of the instruments installed, the extensometers were the most useful. The biggest reasons that these instruments worked so well are because they are robust and very simple in their operating principle. This is in contrast to instrumented rock bolts which can be very sensitive to construction technique, the materials used, the quality of their construction, the quality of their anchorage with the rock, and the quality of their installation in general.

The Kiruna bolts worked poorly as instrumented bolts in this installation. They were apparently susceptible to point loading as was seen by the mismatch between the extensometer deformation and the strain recordings. The D-bolts were not immune to error, either. Data interpretation was complicated by the fact that the locations of the D-bolt anchors and the extensometer anchors were not the same; the D-bolts were anchored at approximately 0, 0.59, 1.34, 2.12 and 3.00 m, while the extensometers were anchored at 0, 1.00, 2.00, and 3.00 m. Thus, the bolts and the MPBX cannot be directly compared, though they can be used to identify trends in movement and loading.

The Kiruna bolt’s rebar texture and full-length cement anchoring made strain readings difficult, but may also have been responsible for improving its rock support performance. Error! Reference source not found. shows that the Kiruna bolts allowed the least separation between the shotcrete and the rock and Table 5 illustrates that less deformation was allowed compared to the same instrumented positions with the other two bolts. The Kiruna bolt allowed less deformation in each location except for the right shoulder, where there was only 0.08 mm difference between the three deformation values.

Table 5 Overview of extensometer deformation by bolt type and instrument location. The lowest and highest absolute deformations allowed in each instrument location are identified in green and red.

<table>
<thead>
<tr>
<th>All values in millimetres</th>
<th>Kiruna</th>
<th>Swellex</th>
<th>D-bolt</th>
<th>Average deformation</th>
</tr>
</thead>
<tbody>
<tr>
<td>Left wall</td>
<td>14</td>
<td>21</td>
<td>17</td>
<td>17</td>
</tr>
<tr>
<td>Left shoulder</td>
<td>2.5</td>
<td>4</td>
<td>5</td>
<td>3.8</td>
</tr>
<tr>
<td>Roof</td>
<td>-1.2</td>
<td>3.5</td>
<td>2</td>
<td>1.4</td>
</tr>
<tr>
<td>Right shoulder</td>
<td>-2</td>
<td>1.5</td>
<td>-1.2</td>
<td>-0.57</td>
</tr>
<tr>
<td>Right wall</td>
<td>-1</td>
<td>10</td>
<td>5</td>
<td>4.7</td>
</tr>
<tr>
<td>Average deformation</td>
<td>2.5</td>
<td>8</td>
<td>5.6</td>
<td></td>
</tr>
</tbody>
</table>

5 Conclusions

Monitoring results in the tested crosscut indicate that the opening is experiencing a stress-driven squeezing with stresses running diagonally between the top left and the lower right corners of the entry. Clear convergence is found in the walls, while extension is found in the right shoulder (Figure 7). This instrumentation could not indicate if tensional failures were occurring in the left floor.

Blasting-induced stresses impacted both the extensometers and instrumented bolts. The concept of impact distance can be used to make a comparison regarding the relative stability of different parts of the entry and can be used with any type of instrument. In this case, the right shoulder had the greatest impact...
distance, indicating that it was most sensitive to stress changes. The left and right walls, as well as the left shoulder, had middling impact distances. The roof had the shortest impact distance, indicating it was least affected by stress changes.

While impact distance indicates sensitivity to stress, it does not account for deformation magnitude. The right shoulder was most sensitive to stress change, but it also had the lowest absolute deformation. Just because a location is sensitive does not mean it is prone to failure. Future research may one day identify a pattern between impact distance and deformation magnitude, though that has not been a goal of this research.

Bolt data indicates that of the three bolt types, the Kiruna bolts performed best in this trial. They offered the least deformation and the least separation between the shotcrete and the rock. Both factors can contribute to the formation of a safe and stable work-place. That being said, this analysis is a snapshot of the conditions prior to the completion of mining on the level above – mining has not yet been completed on the right side of the instruments. The total deformation magnitude experienced in the monitored area has been small compared to other areas in the mine with the biotite schist. The overall conclusion may be a function of the incomplete mining and may change in time.

6 References


Quality assessment of backfill performance for an underground iron mine in Turkey

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Abstract

A hybrid underground mining method is selected for Karacat iron orebody located mid-south east of Turkey. For production levels at higher elevations above 1497, sublevel stoping with stope dimensions of 10 x 25 m is proposed. Below this level, production is planned to proceed with 5x5m cut & fill stopes. Production sequence is arranged for primary and secondary stopes in the sublevels of upper levels. Rock mass properties for the orebody and host rock units were characterized by geotechnical investigations. Results of laboratory experiments and classification work provided input data for numerical models. Considering technical and operational limitations and orebody geometry, various backfill scenarios were modelled. Alternative fill characteristics were investigated to sustain global stability. A relatively high quality fill with an unconfined compressive strength (UCS) of 1.5 MPa for primary stopes, and a lower quality backfill with a UCS of 0.75 MPa for secondary stopes were computed to be sufficient for the stability. For verification of the backfill quality, core samples collected from a field practice were tested in the laboratory. Results of uniaxial compression tests illustrated wide scatter. Observations pointed out that non-uniform particle size distribution with extremely large particles caused insufficient cementation in the backfill. An aggregate mixture with -30 mm particle size distribution and 6-8% cement ratio was recommended for the primary fill. To improve the performance of the filling in practice, waste rock to be used in filling practice was suggested to be processed by a crusher to assure the proper particle size distribution before mixing with cement.

1 Introduction

Large volumes of excavated spaces are left behind as a natural result of underground mining activities. Disturbance of initial stress field in the rock mass due to excavation may lead to global instability problems. Deformation in rock mass provides relaxation for the rock mass stress distribution compared to the induced stresses right after the excavation. Deformations may exceed the limits of elastic rock behavior as a consequence of unsupported volumes of relatively large excavations. Plastic failures fill these voids at the expense of underground mine global stability or surface subsidence in the topography. Such occurrences may slow down or completely halt ore production. Although caving methods are attractive in terms of low production cost, safety concerns may dominate the underground mining method selection. Therefore, supported mining methods might be the first choices of mining companies when global mine safety becomes questionable.
There are two major performance expectations from the backfilling practice in this underground mine. Backfilling is expected to guarantee the stability of individual local stopes during production at different levels. A reliable performance is to be provided by the fill support system during the production throughout the life of the mine. Second expected performance from the filling practice is to maintain a globally safe structural state for the whole orebody, as excavation operations come close to the final stages of stoping and production work in the mine.

Commonly used combinations for backfills of different type include dry sand and rock fills, non-cemented and cemented hydraulic fills, cemented waste-rock fills, paste fills of processing plants, pneumatic fills and fluent fills (Hambley, 2011). Considering the composition, application procedure, and particle size, each of them has advantages and disadvantages. The simplest and mostly preferred choice is to employ dry sand and rock fill. For this, waste rock of the mine from the completed stopes is directly applied with no need to reduce particle size and no cement mixture. Transportation is the single cost item. Low stiffness of this type of backfill is the main disadvantage. Another widely used filling practice includes cemented rock fills. Low strength problem of rock fills is treated by adding a cement mixture. Particle size distribution dramatically affects the quality and performance of cemented rock fills. Backfill composition with coarse size particles may lack planned strength due to cementation problems as a result of low particle surface area in contact with the cement mixture.

For Karacat region, 3D orebody model and DTM (digital terrain model) of topographical surface are identified and extracted by using the field data in the related simulation program. Massive type iron orebody is surrounded by smaller orebodies. For planned underground iron mine in Karacat-Kayseri/Turkey region, backfill design following the stoping sequence is assessed.

Results of rock mass quality characterization work and laboratory experiments are used to estimate the input mass parameters to be used for the stability analyses by numerical modelling. Economical, practical and structural considerations revealed two backfill types as appropriate for primary and secondary stopes. A hybrid mining method with combination of sublevel stoping and cut & fill stoping is decided to be applicable stably with the recommended backfill characteristics.

Unconfined compressive strength tests were conducted on core samples of filling material (containing particles above 30mm and 6-8% cement mixture) collected from field practice of early filling operations. Purpose was to check whether the planned quality of the backfilling support system was being satisfied as recommended in planning.

2 Mine location and general descriptions

Karacat underground mine is located right below the open pit (see Figure 1). Apparently, open pit slope stability will be risky if proper filling design is not applied in underground mining operation. Three critical items of underground mine design are investigated: stope dimensioning, pillar design and backfill design. As part of the overall project, production stope dimensioning is first conducted. Later, pillar stability work is carried out for the multiple stope production operations at different levels of mine. Finally, economically optimum backfill alternatives are assessed from the point of local and global structural stability by numerical modeling.
2.1 Location

Karacat mine is located in Yahyali district of Kayseri / Turkey. A satellite view of the mine, plan view from the 3D model showing the orebody, orebody dimensions and production levels from the cross section view can be seen in Figure 1. Besides from the Karacat mine, there are four active mines: two open pit and two underground mines in the area.

Figure 1 Location of the mine

2.2 Geology

Geology of Karacat iron orebody was studied by Tiringa (2009) in the scope of a Master of Science work. The Geyikdag unit was described to be located in the Taurid Tectonic Belt hosting the Karacat iron orebody. The orebody was reported to be surrounded by Emirgazi (Precambrian), Zabuk (Lower Cambrian), Değirmentaş (Middle Cambrian) and Armutludere (Ordovician) formations.

Hematite and goethite are the major ore minerals which are believed to have originated as a product of siderite alteration. The ore body and country rocks interrelation (Zabuk formation, Değirmentaş formation and Armutludere formation) can be stated to be controlled by tectonism.

Surface reaction mechanism and karstification processes are the result of Post-mineralization faults. Altered siderite and iron minerals transform into limonite and goethite predominated by atmospheric conditions where surface reaction mechanisms are active. Products of the mineralization process mentioned above are exploited and served to industry as raw material.
Karacat Iron ore deposit can be described as a deformed deposit occurred by flow of hydrothermal fluids from Precambrian aged primer iron deposits.

3 Geotechnical studies

2D plane strain numerical modelling is carried out for pillar stability and backfill design. Input parameters of numerical models are decided based on the field studies and laboratory tests. Rock mechanics tests and empirical classifications are conducted by mainly logging and testing on drill core samples.

3.1 Field studies

In the scope of a field trip, rock mass characterization, discontinuity mapping and sample selection for laboratory testing tasks were completed. It was observed that the main ramp excavation face was completed up to the 1497 level. Also, some preliminary stopes were driven.

Calcschist and limestone were observed dominantly both in the hanging wall and foot wall. Ore minerals were seen to be in the form of hematite, goethite, siderite and alterations like limonite, which appeared relatively weaker at the first sight. Empirical classification oriented characterization of rock mass was done both on samples in core boxes and at excavation faces of outcropping parts. Appropriate specimens for laboratory testing were collected.

3.2 Laboratory tests

In METU (Middle East Technical University) Rock Mechanics Laboratory, uniaxial compressive strength tests, triaxial compression tests, static deformability tests, dry unit weight tests and indirect tensile strength (Brazilian) tests according to ISRM suggested methods are carried out on intact rock samples obtained from borehole core specimens. Summary of laboratory test results are presented in Table 1.

Table 1 Laboratory test results of intact rock specimens

<table>
<thead>
<tr>
<th>Test</th>
<th>Ore</th>
<th>Hanging Wall - Foot Wall</th>
</tr>
</thead>
<tbody>
<tr>
<td>Unconfined compressive strength, UCS (MPa)</td>
<td>45.5</td>
<td>89.8</td>
</tr>
<tr>
<td>Young modulus, E (GPa)</td>
<td>12.6</td>
<td>21.1</td>
</tr>
<tr>
<td>Poisson’s ratio, v (MPa)</td>
<td>0.17</td>
<td>0.11</td>
</tr>
<tr>
<td>Cohesion, c (MPa)</td>
<td>11.4</td>
<td>10.1</td>
</tr>
<tr>
<td>Internal friction angle, φ (°)</td>
<td>51.3</td>
<td>53.5</td>
</tr>
<tr>
<td>Tensile strength, σt (MPa)</td>
<td>4.8</td>
<td>9.8</td>
</tr>
<tr>
<td>Unit weight, γ (kN/m³)</td>
<td>29.4</td>
<td>27.1</td>
</tr>
</tbody>
</table>

3.3 Rock mass characterization

The GSI system was used to characterize rock mass exposures on the excavation faces in the field. RMR<sub>89</sub> (Bieniawski, 1989) and Q system (Barton, 1974) are used to define rock mass quality through a depth of 30 m in the hanging wall and a depth of 20 m depth in the footwall. The reason for concentrating on these critical depth bands in the hanging and footwall can be explained by their...
effectiveness in the local and global stability of the mine. Shear zones located within the orebody are about 3-5 m thick. Shear zones in the hanging wall and footwall are 1-3 m thick. All of these zones show repetitions at every 20 to 30 m. Average ratings and quality descriptions can be seen in Table 2.

Table 2  Average rock mass quality scores with respect to GSI, RMR and Q systems

<table>
<thead>
<tr>
<th>Rock Unit</th>
<th>GSI</th>
<th>RMR&lt;sub&gt;89&lt;/sub&gt;</th>
<th>RMR Quality Desc.</th>
<th>Q</th>
<th>Q Quality Desc.</th>
</tr>
</thead>
<tbody>
<tr>
<td>Hanging Wall</td>
<td>53</td>
<td>58</td>
<td>Fair</td>
<td>4.2</td>
<td>Fair</td>
</tr>
<tr>
<td>Ore</td>
<td>45</td>
<td>49</td>
<td>Fair</td>
<td>3.4</td>
<td>Poor</td>
</tr>
<tr>
<td>Footwall</td>
<td>40</td>
<td>45</td>
<td>Fair</td>
<td>1.0</td>
<td>Poor</td>
</tr>
</tbody>
</table>

4  Backfill design and quality assessment

Underground mine stability is critical even after the exploitation is completed. Surface subsidence triggered by underground activities can lead to serious risks for the overlying abandoned pit slopes. This instability in turn can present a major problem for underground workings. In addition to protecting the neighboring local stopes, backfilling has become a standard cycle of underground mining in spite of its increasing effect on the mining cost.

Numerical models confirmed that production below 1497 level of the mine with stoping techniques dramatically affected global mine stability. Therefore, a hybrid underground mining method was considered and analyzed. Sublevel stoping method was planned to be applied above 1497 level. Below this level, ore would be extracted by cut & fill method.

For primary and secondary stopes, two backfill alternatives are studied. Effect of backfill quality on stability of local orebody pillars is analyzed as the first issue. Towards the end of the production in a mine level, a few stopes which are not extracted yet, behave as only supporting orebody pillars. They are overloaded and contain the risk of pillar burst due to high stress concentrations. FEM models are constructed to analyze this risk. At the final stage, completely filled underground mine stability is analyzed for checking the global stability state.

4.1  Modeling entries for stability analyses

Numerical analyses are carried out in Phase2 Finite Element Method (FEM) software with the assumption of plane strain condition. FEM models are prepared on two critical cross sections across the orebody using 3D topographical surface and orebody solid model (Figure 1).

Rock mass properties are estimated by using GSI values and intact rock UCS values in ROCKDATA software. Output is a fitted curve for the Hoek&Brown failure criterion given in equation (1). From the Generalized Hoek&Brown failure criterion (Hoek et. al., 2002,) it is possible to estimate equivalent Mohr-Coulomb parameters as cohesion and internal friction of rock mass.

\[
\sigma_1 = \sigma_3 + c_{ci}\left(m_b \frac{c_s}{4\epsilon_t} + s\right)^a
\]  

Where \(m_b\), \(s\), and \(a\) are well-known constants representing the different structural states of the rock mass.
For plastic analyses, GSI scores representing residual states are calculated from equation (2). Using laboratory tests and regular GSI scores, residual strength parameters are calculated based on the residual GSI given in equation (2) by Cai et al., 2007.

\[
GSI_{res} = GSI e^{-0.0134 GS1}
\]  

(2)

Peak material properties represent the transition from elastic to plastic material behavior. It aims to examine initial failure in structural units. However, residual material properties focus on the state after failure with increasing deformations. For instance, plastic material volume increases after failure due to broken rock pieces and this leads to greater deformations at the excavation boundaries. Volume increase is controlled by dilation angle parameter. Rock mass dilation angle in modeling work is imposed based on the results of equation (3) suggested by Alejano et al. (2009):

\[
\psi = (5GSI - 125) \phi / 1000
\]  

(3)

For the residual rock mass tensile strength, ratio of tensile to compressive strength obtained from the laboratory tests is adopted. This ratio is around \( \sigma_t / \sigma_c = 0.1 \) for the laboratory strength; so it is kept the same to estimate the ratio of rock mass strength entries in the models. Predicted rock mass tensile strengths based on Hoek & Brown criterion are too low and cause instabilities in model integrity.

In Table 3, rock mass parameters for peak and residual states of rock units, and contact zone are given. Although backfill alternatives with varying mechanical properties are investigated in the models, only the ones considered in final decision is presented in Table 3. Primary backfill strength (particle size below 30 mm and 6-8 % cement mixture) is suggested be around at least 1.5 MPa. Secondary backfill is estimated to be satisfactory with a strength which is about half of the primary fill.

### 4.2 Results of modeling work

Above the 1497 level, 10x25 m stopes are planned to be applied. Primary and secondary stopes are proposed to be filled with the appropriate backfill materials. Below the 1497 level, 5x5 m cut & fill stopes are planned to be located; and all the stopes here are suggested to be filled with primary backfill material. Effect of backfill material on local stability of stopes and on the global stability of the mine are analyzed in detail.

Elastic models are generated to compute the safety factor of the pillars that are locally protecting the production stopes. Stress distribution in pre-failure state will be greater compared to the post failure of a pillar. Thus, elastic model that represents the most critical state (pre-failure state) works better to predict the pillar safety factor. Data points of related structural entries are presented in the model outputs along the model query lines across the pillars. Using the principal stresses of the model outputs, rock mass strength state is estimated by comparing cohesion and internal friction angle based failure criterion to the stress state given by model outputs.

Equation (4) is for the computation of rock mass pillar strength at varying confinement stress throughout a typical pillar:

\[
\sigma_{1\text{strength}} = \sigma_{c\text{mass}} + q\sigma_3
\]  

(4)
Table 3  Input rock mass properties for numerical analyses

<table>
<thead>
<tr>
<th>Rock mass units</th>
<th>Ore</th>
<th>Hanging Wall</th>
<th>Foot Wall</th>
<th>Contact Zone</th>
<th>Primary Backfill</th>
<th>Secondary Backfill</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Peak Res.</td>
<td>Peak Res.</td>
<td>Peak Res.</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>GSI</td>
<td>45 25</td>
<td>53 26</td>
<td>40 23</td>
<td>25</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Unit weight (kN/m³)</td>
<td>29 4</td>
<td>27 1</td>
<td>27 1</td>
<td>27 1</td>
<td>20.0</td>
<td>20.0</td>
</tr>
<tr>
<td>Modulus of Elasticity (GPa)</td>
<td>2.8 1</td>
<td>7.71</td>
<td>3.36</td>
<td>0.75</td>
<td>0.60</td>
<td>0.30</td>
</tr>
<tr>
<td>Poisson’s ratio</td>
<td>0.17</td>
<td>0.11</td>
<td>0.11</td>
<td>0.17</td>
<td>0.30</td>
<td>0.30</td>
</tr>
<tr>
<td>$\sigma_{c,\text{mass}}$ (MPa)</td>
<td>11.2 7.0</td>
<td>30.7 17.1</td>
<td>23.8 15.9</td>
<td>7.0</td>
<td>1.5</td>
<td>0.75</td>
</tr>
<tr>
<td>Tensile Strength (MPa)</td>
<td>1.1 0.7</td>
<td>3.1 1.7</td>
<td>2.4 1.6</td>
<td>0.7</td>
<td>0.2</td>
<td>0.1</td>
</tr>
<tr>
<td>Internal friction angle (°)</td>
<td>37 31</td>
<td>42 34</td>
<td>39 33</td>
<td>31</td>
<td>35</td>
<td>35</td>
</tr>
<tr>
<td>Cohesion (MPa)</td>
<td>2.8 2.0</td>
<td>6.8 4.5</td>
<td>5.7 4.3</td>
<td>2.0</td>
<td>0.4</td>
<td>0.2</td>
</tr>
<tr>
<td>Dilation angle (°)</td>
<td>4 6</td>
<td>3 3</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>

Where $\sigma_{c,\text{mass}}$ is global strength and $\sigma_{1,\text{strength}}$ is rock mass compressive strength under confinement. Here $q$ is confinement stress factor that can be calculated from internal friction of rock mass.

Pillar walls remain unconfined unless support is applied. Instead of using rock mass uniaxial compressive strength ($\sigma_c$) suggested by ROCKDATA processing, global strength ($\sigma_{c,\text{mass}}$) is preferred for computing FOS. Global strength is preferred to be used in comparison to the major principal stress for pillar wall stability investigations around the underground excavations. Direct use of $\sigma_c$ is not advised, since it suggests values which are too conservative for pillar design applications. Instead, use of global strength in Equation (5) is recommended by Hoek&Brown (1997) for stability checks at pillar walls.

$$\sigma_{c,\text{mass}} = \sigma_{c,\text{strength}} \frac{(m_b+4s-a(m_b-8a)(m_b/4+a)^{a-1})}{2(1+a)(2+a)}$$  \hspace{1cm} (5)

Inside pillars where confinement exists, $\sigma_{1,\text{strength}}$ of Equation (4) is used to estimate factor of safety as in equation (6):

$$FOS = \frac{\sigma_{1,\text{strength}}}{\sigma_1}$$  \hspace{1cm} (6)

In a stope & pillar type mining, acceptable factor of safety is around 1.3 for local and global stability of the mine. In this study, models are constructed with 2D plane strain assumption. With this assumption, pillars are expected to be loaded more compared to the 3D structural state. So, plane strain modeling remains on the safe side. Thus, any FOS greater than 1.2 for pillars of 2D modeling is predicted to indicate that those pillars stay sufficiently on the safe.
Plastic analyses in modeling work yield failure types (like shear or tensile failures), failure zone and geometry. This way, it is possible to comment on the effectiveness of any support type depending on the failure mode.

Between the orebody and hanging wall, a shear zone of around 3-5 m thick is detected from the drill core sample observations. This part is named as contact zone and orebody geometry is encapsulated by such a zone in the models.

In order to check the validity of the calculated $\sigma_{c_{mass}}$, existing trial stopes are modelled and interpreted using back analysis method. Current failure zones in opening walls and model results are compared. In Figure 2 total displacement contours and failure zones in a plastic analysis can be seen. From the field investigations it is roughly estimated that depth of failure zones is 600 mm in walls and 10 mm in roof. Compared to the numerical model it is concluded that calculated rock mass properties are representative for the field.

**Figure 2** Total displacements and failure zones around existing preliminary stopes

In the numerical models, production sequence starts from the orebody below 1497. Cut & fill stopes with dimensions of 5x5m are planned. Maximum four stopes are produced at the same time and the safety distance between operating stopes is 30 m. Production starts from the lower level that is 1477. Cut & fill stopes are completely exploited and filled till 1497. Later, sublevel stoping method is applied above 1497 level. First, primary stopes are produced and filled with primary backfill. Filled primary stopes behave just as pillar while secondary stopes are produced. Secondary stopes that are completed are filled with secondary backfill material. Section view of the model showing the end of production sequence can be seen in Figure 3. A contact zone presenting the deformed layer on the orebody – wall rock contact can also be seen in this model.
Backfill stability is examined separately at mine levels below and above the level 1497. Firstly, primary backfill performance in cut & fill stopes below level 1497 during production is analyzed. End of production sequence below level 1497 is modelled next.

Stability of primary stopes are investigated for conditions at during production and at end of production stages above level 1497. Last stage of stability assessment for mining for above the level 1497 is for the secondary stopes. Results of numerical models are processed for during production and end of production stages here.

Finally, any subsidence effect induced instability of completely filled underground mine on topographical surface involving the deep open pit is analyzed.

In figure 4, total displacements and failed elements at the end of the production below 1497 can be seen. Orebody settles on the backfilled cut & fill stopes with a deformation of around 270 mm. It is observed that backfilled zones decrease the induced stresses generated by production openings.
Above 1497, 10x25 m sublevel stopes are planned. In the first critical scenario model of the stope layout above 1497, most of the primary stopes are completed and filled. The structural layout for the first critical scenario can be seen in Figure 5. This case simulates the production phase at which maximum load is transferred to the pillar marked in the figure. As can be seen, average factor of safety throughout this pillar is around 2.5. Even for the most critical state, pillar safety is highly satisfactory. Apparently, backfilled stopes decrease the stress concentration on the orebody pillar and increase the factor of safety of this pillar.

Figure 5   Structural analysis layout for the critical sequence – scenario 1

Another critical state is analyzed by a scenario for the first stoping level above 1497 in which production and filling is completed for all primary and secondary stopes, except the last production stope. Adjacent to this last stope, both sides of the stope consist of primary filling. In Figure 6, plastic model and solution results can be seen. Displacement magnitudes at the stope roof are seen to be in the order of magnitudes less than 100 mm levels. Considering the stope dimensions in orders of tens of metres, total displacements on the stope roof can be regarded insignificant for the overall stability of the stope. Thus, it can be stated that primary and secondary fills around this last stope are effectively performing supporting action as planned.
Figure 6  Structural analysis layout for the critical sequence – scenario 2

At the upper levels, considerable increases in the pillar safety factors are observed and plastic analysis results do not point out any critical deformation states around stopes of the upper levels.

Figure 7 represents the surface subsidence after the mining operations are completed and stopes are filled completely. Maximum subsidence is around 140 mm. It is concluded that this much deformation magnitude is not significant for global stability of the mine which has dimensions over 100 metres and depth around 120 m.

Figure 7  Surface subsidence at the end of backfilling operations and mine life.

Backfill strength is directly related with the cement ratio and aggregate rock strength. In order to provide the global mine stability presented with the numerical models above, aggregate rock particle size should not increase 30 mm and cement mixture should be used as a binder. If the waste rock strength is low, high strength aggregates like limestone should be used.

4.2  Evaluation of backfill performance in practice and recommendations

In mining practice, production started from the bottommost level. After completing a few of stopes, initial backfilling attempts were carried out under control by the company’s technical staff. Drill cores were taken from a sample primary backfill operation site for mechanical tests. In coring process, low core recovery was reported. This meant that filling practice was not effective and there were quite
large unfilled spaces in filling locations. Drill core samples with 63 mm diameter taken form parts through the waste and cemented mixture were tested in the Rock Mechanics Laboratory.

Visual inspections showed that particle size exceeded the recommended -30 mm suggestion. Another issue was identified as the irregular distribution of cement mixture. Improper cementation process was not able to construct strong enough binding among the rock particles and cement. Binding was not sufficiently tight and large voids remained inside.

Overall, eight uniaxial compressive strength (UCS) tests were done on cores of this filling texture. Table 4 shows the test results. As can be seen, standard deviation of UCS is high. While maximum UCS is 9.2 MPa minimum UCS is 0.9 MPa. High UCS results correspond to the testing of cores made up of stone boulder parts of the waste. Observations indicate that some core samples are directly taken through the rock and large boulder parts filling the stopes. Thus, test results shown below should not be used to represent large field scale back fill strength characteristics. Instead, core samples taken from backfill with proper particle size distribution and cement mixture should be used to investigate the backfill quality.

Aggregates can be obtained from waste rock extracted from the mine. Maximum particle size should be reduced below 30 mm by passing them through crushers. Performance of secondary backfill supplied from the waste rock is analyzed in the modeling work, and it is concluded that if the contact between the backfill upper level and the roof can be sustained with proper backfilling and sufficient yielding of the roof, there is no need to add cement mixture.

For primary filling practice, recommendation is to crush the waste rock down to 30 mm with an onsite crusher. In addition, aggregate and cement should be thoroughly mixed before filling into the stope. The mixture should be uniformly distributed, and it should entirely cover the stope space. Although no strength problem for the stone parts of the aggregate is observed, high quality aggregates like limestone available at mine site are suggested to be used in case of low waste rock quality.

Table 4 Laboratory test results

<table>
<thead>
<tr>
<th>Sample No</th>
<th>Density (kg/m³)</th>
<th>Average Density (kg/m³)</th>
<th>UCS (MPa)</th>
<th>Average UCS (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Spc1</td>
<td>2.3</td>
<td></td>
<td>4.1</td>
<td></td>
</tr>
<tr>
<td>Spc2</td>
<td>2.2</td>
<td></td>
<td>6.3</td>
<td></td>
</tr>
<tr>
<td>Spc3</td>
<td>2.3</td>
<td></td>
<td>0.9</td>
<td></td>
</tr>
<tr>
<td>Spc4</td>
<td>2.1</td>
<td>2.2±0.1</td>
<td>3.5</td>
<td>4.2±2.7</td>
</tr>
<tr>
<td>Spc5</td>
<td>2.0</td>
<td></td>
<td>4.9</td>
<td></td>
</tr>
<tr>
<td>Spc6</td>
<td>2.4</td>
<td></td>
<td>9.2</td>
<td></td>
</tr>
<tr>
<td>Spc7</td>
<td>2.3</td>
<td></td>
<td>3.0</td>
<td></td>
</tr>
<tr>
<td>Spc8</td>
<td>2.2</td>
<td></td>
<td>1.4</td>
<td></td>
</tr>
</tbody>
</table>
5 Conclusion

Backfill design and quality control attempts in Karacat underground iron mine are analyzed. Field investigations and laboratory tests are used to define rock mass characteristics and mechanical properties. Stopping methods are primary choice of any company to lower the production cost. Numerical studies revealed the instability in case of production with a single method, which is sublevel stopping. Lower rock quality and increasing stresses with depth around the pillars and stopes of the footwall levels are identified as the main reasons of the instability.

A hybrid underground mining method is proposed and analyzed to ensure stability of local pillar-stope layouts and global mine stability. Below the mine level 1497, production starts by cut & fill method. Completed stopes are filled with the primary type of fill. Above the level 1497, sublevel stopping method with larger stopes is suggested to be applied, considering the economic feasibility of large stopes in mining practice.

Observations and mechanical tests were conducted on backfill samples from the field practice. It was observed that aggregate particle size of mine filling applications was much greater than the recommended maximum size of -30 mm. Because of large particle size, cement mixture was concluded to be ineffective in binding the rock particles properly. Low core recoveries reported during core drilling through the filled areas implied that there were large voids inside the supposedly filled stopes.

For primary stopes, cemented rock fill is recommended. UCS of this fill is expected to be around 1.5 MPa. To achieve the desired characteristics, mine waste rock is recommended to be crushed below 30 mm particle size and mixed with a cement ratio of 6-8%. For secondary stopes, a rock fill with a UCS of 0.75 MPa is found to be satisfactory.

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References


Real-time risk assessment and ground support optimisation in underground mines

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Abstract

Underground mines rely on point stress measurements, which are limited both in time and in space. Developing a stress monitoring system that can provide real-time information about the surrounding rock mass stresses is seen as highly prospective. Mines could quantify the level of rock mechanical hazards and track stability of the excavations. A method for real-time stress change estimation was developed at Aalto University. The method uses strain measurements, which are obtained using extensometers and inverse calculation of stress change using superposition of unit stress responses. It was tested in Boliden’s Kylylahti mine. Aalto University developed guidelines for risk assessment in underground mines that can utilize real-time rock stress input in order to quantify the level of geotechnical hazards. The long-term goal is to develop a concept for real-time monitoring and risk assessment in underground mines. The objective of this paper is to summarize the preceding research and to investigate how the real-time in situ stress data can be used for real-time risk assessment and ground support optimisation in underground mines. The key concepts are introduced and the required changes to the process with real-time data are described. An example from Kylylahti mine is presented as an example use case. The current state, required changes, advantages and difficulties with the current approach are discussed. Finally, based on the observations, suggestions for future research are given.

1 Introduction

In recent years the authors have developed mining safety research at Aalto University in two projects. The first is a project for the i²Mine (Innovative Technologies and Concepts for the Intelligent Deep Mine of the Future) under the 7th framework programme of the European Union, where a formal geotechnical risk management guideline was developed to tackle the geotechnical risks in underground mines (Mishra, 2012; Mishra & Rinne, 2014; Janiszewski, 2014; Janiszewski et al., 2015). In the second, Dynamine project under Tekes Green Mining programme, the objective was to create a real-time rock mechanical monitoring concept for the mining industry (Ritala, 2015; Ritala et al., 2016; Kodeda et al., 2015). The goal was to increase the safety of deep underground mines where large changes in the stress field due to stope excavations can cause significant risks. In this paper the two concepts, the guidelines for the developed geotechnical risk management and real-time stress change monitoring, are joined to study the potential to provide a solution for real-time risk assessment and ground support optimisation in underground mines.

The long-term goal is to develop a concept for real-time monitoring and risk assessment in underground mines. The objective of this paper is to summarize the preceding research and to investigate how the real-time in situ stress data can be used for real-time risk assessment and ground support optimisation in underground mines. The current concept is based on stress state change estimation algorithm based on back-calculation of measured strains. The real-time monitoring is able to provide feedback about the success of mining sequencing and sufficiency of ground control methods. By monitoring the actual change,
the design and the sequencing of the stopes can be turned into an iterative process. With the real-time monitoring, it is possible to increase the safety of underground mines if the stress changes cause risks.

In the geotechnical risk management (Figure 1), the geotechnical hazard potential (GHP) is an evaluation performed using an indicative ranking of mining operations based on the potential it has in causing a geotechnical hazard. Modified Barton’s Q value is used to classify rock mass competency into 5 classes ranging from ‘very good competency’ to ‘very poor competency’. The GHP classification system can be translated into a preliminary risk assessment to justify a formal hazard-specific risk assessment for an area to predict and prevent geotechnical accidents. Next, the geotechnical risk assessment is executed to identify and mitigate hazards before they pose risk to the working environment. The risk assessment process has been divided into five phases:

1. Outlining the scope of risk assessment
2. Identification of hazards within the scope
3. Evaluation of the likelihood of hazard
4. Assessment of the consequences (exposure to hazard)
5. Ranking the risk to formulate a strategy of risk reduction.

Figure 1 Geotechnical risk management flowchart (Janiszewski et al., 2015)
If the level of risk is known for a particular location, then mitigation measures are selected and implemented in order to reduce it to an acceptable level. Reliability assessment aims at estimation of the degree of confidence the mitigation measures provide against the risk. Monitoring is an important part of geotechnical risk management, as it provides information that can be used in quantification of hazard probability during the risk assessment process, and is used for review and evaluation of the reliability of mitigation measures. Geotechnical risk management is a cyclic process, where all activities are repeated systematically until all risks are reduced to an acceptable level. Communication and consultation of results with all involved partners is crucial in order to facilitate good transfer of knowledge and hazard data through different stages of mining.

There are several geotechnical hazards that are caused by the extensive deformation of rocks. Rockfall is an uncontrolled detachment and movement of rock fragments into the excavation. Rockfalls cause 24% of fatal accidents in underground mines worldwide (MacNeill, 2008). Real-time monitoring of the movement of excavations can be used for evaluation of rockfall hazard. An example of such system has been presented by Vogt et al. (2010), where an AziSA standard is applied for rockfall early warning systems in South African mines. Data from measurements of excavation closure is combined with thermal images and acoustic sounding tool data to produce a rockfall risk map for different areas in the mine. Another type of geotechnical hazard, where monitoring of rockmass conditions is used for hazard assessment, is rockburst, a violent failure of rock due to high concentrations of stress around the excavation. It is often present in deep mines in hard and brittle rocks. High rockburst hazard is associated with high rock stress conditions and rapid changes in stress which may lead to a seismic event. Rockfalls and rockburst can cause failure of ground support and damage the excavation, hence resulting in loss of its functionality.

Use of the real-time monitoring is best accompanied with the observational method that allows a controlled way to adapt to the geotechnical behaviour. The method, first suggested by Terzaghi and defined later by Peck (1969), is based on establishing plans in advance, monitoring the ground behaviour during excavation and then executing contingency actions to select the ground support based on the measured ground behaviour during excavation. In this way the conservativism of the plans can be decreased while still achieving safe and economical ground support. The relevant hazards are identified in the planning stage using the geotechnical risk management system. The combination of the geotechnical risk management system, observational method and real-time monitoring will rationalize ground support measures in challenging conditions.

2 Real-time risk management

The philosophy behind real-time risk management and monitoring of rock mass can be described by the Data-Information-Knowledge-Wisdom (DIKW) hierarchy proposed by Ackoff (1989). ‘Data’ represents raw measurements and ‘Information’ is the data, which acquires meaning through identified connections and relationship with other data. ‘Knowledge’ is the collection of information, which enables understanding of patterns. ‘Wisdom’ is created by applying the knowledge through evaluated understanding and represents the decisions made by humans. The DIKW hierarchy is plotted on Figure 2 as a chain of increasing connectedness of links in the chain (Y-axis) and our understanding of the process (X-axis). Knowledge is all data and information from the events that took place in the past, and wisdom represents the predictions made about future events. In relation to real-time monitoring, the DIKW hierarchy illustrates how increasing the collection of information by more frequent recording of data can be used to create more knowledge and make better predictions about future events and their effect on the performance of excavations.
Risk is a product of the probability of a hazard to occur and the consequences (or severity) that might arise if the hazard takes place. Some risk assessment methods include the vulnerability of an excavation in the probability part of the risk equation, which determines its proneness for damages due to large ground movement and deformations. An example of vulnerability concept is the Rockburst Damage Potential (RDP), which is an empirical index proposed by Heal et al. (2006) for estimation of rockburst damage, and is implemented in the Mine Seismicity Risk Assessment Program. The RDP relates the peak ground movement and several factors, such as stress conditions, ground support capacity, excavation span and geological structure, in order to estimate the susceptibility of an excavation to be damaged during a rockburst event. The consequences part of the risk equation is sometimes represented as the exposure to the hazard to express the quantity of assets that are at risk, which directly correlates with how severe the hazard can be. Given the real time measurements deal with specific risks, the exposure cannot be ignored. According to Woodward & Wesseloo (2015) the influence of stress change is considerable and has a big impact on the probability of a rockburst to occur:

“Elevated rates of seismicity in a mining environment are commonly observed following significant changes in stress conditions, lasting in the order of hours to days.”

Access to real-time information about changes in stress state can provide an indication of elevated seismicity expected to occur and can be a valuable tool for monitoring of the vulnerability of an excavation. Such knowledge can be used in early warning systems and entry protocols to lower the risk in underground mines. Real-time monitoring of rock mechanical conditions can be a valuable tool for proactive maintenance of underground excavations and adjusting reinforcement design during mining operation. Changes in the rock stress field induced by mining can be anticipated and their impact on continuation of operation can be evaluated in real-time. Furthermore, adjustments to mine design can be done based on the results from monitoring, for example selection of favourable orientation of the excavation with respect to the stress direction in changing mining environment.

Real-time measurements eliminate the data acquisition time and therefore reduce the data analysis time. In principle, any geotechnical accident should give yielding signs but they can be missed if the prominent events happen between conventional monitoring intervals. Site specific hazard likelihood assessment models can be built when historical data is available. These models can be then automated to continuously evaluate real time measurements to provide a live view of the hazard states such as stress and seismicity. This reduces chances of human error and thus ensures high reliability. Quick reaction time ensures that the maintenance of an underground opening is non-invasive and corrective actions such as additional reinforcements are done before the failure symptoms actually occur. With the advancement in equipment
automation in deep underground mines, the preference and extent of wireless network coverage is increasing. Existing instrumentation such as extensometers and geophones can take advantage of the existing network infrastructure to transmit real-time data to a central data repository. Even in mines without a wireless infrastructure the need for an effective warning system becomes justifiable with depth and increasing geotechnical risk. Risk can be evaluated in real-time and dynamically with continuous assessment of information. High risk mining areas can be even equipped with a safety-state warning system.

3 Risk assessment process with real-time data

Based on the guidelines developed to carry out Geotechnical risk assessment (GRA), GRA focuses on four primary forms of industry-wide risk assessment methods, namely Workplace Risk Assessment and Control (WRAC), Failure Mode and Effect Analysis (FMEA), Bow Tie Analysis (BTA) and Fault Tree – Event Tree Analysis (FTA – ETA) (Mishra et al. 2014). These 4 methods are misnomers as they primarily provide a framework of arriving at various possible hazards that can lead to a top event but not the likelihood of it happening. These methods help answer “What can lead to an accident?” but do not answer the crucial question of “Will the accident happen?”. Real-time measurements help determine the likelihood of hazard and can be used with any of the four risk assessment methods. Based on the GRA guidelines, the Fault Tree – Event Tree analysis is the preferred choice of risk assessment because it uses quantitative probability of occurrence values which are data driven.

Fault Tree Analysis (FTA) is a deductive method of hazard analysis where a hazardous event is evaluated downwards to identify all the possible causes leading to an event (Iverson et al. 2014). This includes combinations of hazardous conditions which lead to an accident. FTA is comprised of 3 components. First is ‘Top event’ which is the principle accident in consideration such as roof fall. Second is ‘Hazards’, which are the conditions which lead to an accident. Third is ‘gates’ which are Boolean logic operators such as ‘and’ and ‘or’ which imply if all or either of the hazards are needed to be present to cause the accident. This process is done till the primary hazards are identified. Probability of each primary hazard is calculated, which in turn gives the likelihood of the accident. Event Tree Analysis deals with the probability of the various consequences of the top event. Event tree does not comprehensively quantify the full severity of a consequence but helps to narrow down the various possibilities.

An effective risk management system must integrate all available information sources to form the backbone of a robust intervention system. This is done by evaluating all possible failure modes of a system. The analysis is done till all possible rootcauses (leading eventually to the incident/accident) are identified. This exercise not only aids in a detailed probabilistic calculation but also presents a visual map of various ways in which a system can fail and different components that contribute to it.

A hypothetical example is considered in Figure 3 showing a basic fault tree analysis where the top hazard is stresses exceeding the support capability. In this example, the root causes identified are incorrect support design, inferior support quality, incorrect support installation which can lead to undersupporting of a mining area. The other causes identified are unexpected stresses created by incorrect mining sequence and/or blasting/seismicity induced stresses. Once all the root causes or primary hazards are identified, the probability of them occurring is evaluated. Where data is available, probability can be evaluated using historical occurrences. If a mine uses a rigorous quality assurance/quality check (QA/QC) programs and the historical data suggests that less than 5 out of 100 samples were substandard within a given period, the probability of a support being of inferior quality is set at 5%. Incorrect design probability is a combination of quality and extent of data used for the design and the competence of the person/team responsible for the design. While periodic measurements of modeled vs. measured data can provide an estimate of instances a mine gets the design wrong, real-time measurement can provide the exact time when the deviation occurs and prevailing circumstances that led to it. In this example the assumption made is that the chance of poor design is 10%. Chance of incorrect installation can also be obtained using historical data for a mine with QA/QC procedures in place. Additionally, real-time measurement of deviation and follow up investigation of affected area can also help reduce installation errors. The assumed probability of incorrect installation in
this example is 5%. Similarly, historical data can help establish how often does the mine deviate from planned mining sequence, and real-time measurements can help to establish the impact of blasting on the stress state of the mining area. The probability of incorrect mining sequence and blasting/seismicity induced stress change beyond support capacity is assumed to be 5% and 10% respectively. With the above probabilities, the likelihood of a poor support evaluates to 18.8% while probability of mining induced stress exceeding support capacity evaluates to 14.5%. This results in a net probability of the hazard of 31%. However, a real-time measurement of geotechnical parameters can help build an early warning and intervention system. If such a system having a 10% chance of failure between planned maintenance is put in place as a deterrent, the likelihood of the hazard being realised is reduced from 31% to 3.1%. Therefore, with real-time monitoring in place, trends of deteriorating geotechnical conditions can be continuously evaluated and mitigating measures can be put in place well in advance thus reducing the risk of an accident to an acceptable level.

Figure 3 Fault Tree Analysis (FTA) showing hazards leading to stress exceeding support with real-time monitoring as a deterrent

4 Decision making with real-time data

Data processing and analysis can be grouped into three categories depending on the resources used and the complexity of the model.

4.1 Direct correlation between failure and monitored data

First approach uses the direct correlation between failure and monitored data when substantial historical data is collected with similar rock properties, which are then used to assess a new region. Information of rock properties at the time of failure or near failure should be available to build a direct correlation between information being measured and reported, and hazard being assessed. For instance, if sufficient evidence is available to show that when displacement measurement from extensometers is \(x\) and/or seismic reading from geophones is \(y\), it leads to rockburst in level \(z\). Given the variability in rock properties, the ideal process is to set various levels of thresholds to warn if risk is low, moderate or high. This method does not require any data processing and local rugged computers can also be used to change the risk status
of a mining area. One of the key disadvantages of this is that it only looks at the symptoms of failure and does not try to evaluate the state of underlying hazard such as high stress. Given the impact of mining induced variables such as blasting, sequence of excavation on underlying hazards, it is necessary that such a warning system works with high factor of safety.

An example of this approach is the observational method, which is often credited to Peck (1969), although his paper actually is more of a synthesis of a practical work approach called the learn-as-you-go method, which was formulated and developed by Terzaghi during the preceding decades. The benefit of the observational method is the possibility to use measurements to reduce the amount of conservatism caused by geotechnical uncertainties. Peck’s definition has eight steps: exploration, assessment of most probable and most unfavourable deviation, most probable design, selection of quantities to observe, calculation of quantities under unfavourable conditions, selection of course of action for every foreseeable deviation, measurement and evaluation, and modification of design to suit conditions.

The Eurocode 7 (EN 1997-1:2004) implementation of the observational method is straightforward: establish limits of acceptable behaviour, assess range on possible behaviour, plan monitoring, plan contingency actions, monitor. It discards the most probable and most unfavourable approach, but otherwise it is an extension of the method as described by Peck. Spross (2014) criticises the observational method approach presented in EC7, pointing out that it is unclear how to determine acceptable behaviour, the approach apparently lacks a safety margin, the predictability of the control parameter is neglected. He suggests that methodology should be developed to assess the safety of structures built using the observational method, long-term extension of the method for monitoring of existing structures and a link to probabilistic design of structures.

To use the observational method in conjunction with real-time data, some additions to the Eurocode procedure are needed. First, preliminary modelling is used to select locations for the sensor network. Then, detailed modelling is carried out for each sensor to produce unit responses. Multiple linear regression is used to solve the corresponding stress state. These stress states are then compared to the corresponding ground behaviour.

The ground behaviour is numerically analysed and the acceptable limits for the local stress state are stored. Next the unacceptable states are analysed for damage extent and damage rate (e.g. strain rate). Here the risk assessment is in a key role. Sufficiently small and slow damage allows for contingency actions to be put into place. Finally the unacceptable limit is stored. Traffic lights can be used to illustrate the purpose of the three areas and two limits: the green area is when the stress state is acceptable, yellow area implies that contingency actions must be installed and red area indicates failure.

The possible ground behaviour is split into categories. In the green category we have the predetermined and expected design solutions. In the yellow category we have the remedial and contingency actions. The red category implies unavoidable damage. The decision making process is automated. The sensors transmit the readings at regular intervals, the inversion is carried out and the resulting stress state is compared to the behaviour categories. If the green limit is exceeded the site engineer is immediately informed and the system recommends the contingency action appropriate. The reaction time and the installation time must be taken in account. If passive reinforcement is used, the activation deformation must be calculated to avoid collapse (too late installation) and failure of the reinforcement (too early installation). If the yellow limit is exceeded, the workers in the area are evacuated and the access to the area is prevented. This can occur for example if the strain rate exceeds the stable limit.

4.2 The use of predefined algorithm to evaluate underlying hazard

Second approach is based on evaluating correlation between monitored data and underlying hazard. This is carried out by subjecting monitored data to predefined algorithms to arrive at the state of the underlying hazard. An example of this approach is a method developed at Aalto University during the Dynamine project, where solution for real-time stress state change tracking was tested (Kodeda et al., 2015; Ritala, 2015; Ritala et al., 2016). The solution for stress state change tracking is explained in more detail in article
by Kodeda et al. (2015). The basic principle of the solution is to measure strains with extensometers and then to compare the results to modelled strains. Through this statistical comparison the stress state changes can be solved. The analysis method created can be used to create threshold values for ground control management purposes.

The test was done in two different locations at Kylylahti mine site during 2015 (Ritala, 2015; Ritala et al., 2016). The test sites were instrumented using multipoint borehole extensometers (MPBX). The first step of the instrumentation was to model the test site area with future excavations to achieve optimal placement for the extensometers. The area was modelled with Examine2D. It was considered to be important that the extensometers would be placed in the areas where most of the stress-driven strains would occur. Second, the MPBXs were installed and a new model based on the locations of the MPBXs was created. Third, the mining progressed to the next stope and the strains caused by the mining were measured. Fourth, the measured strains were compared to modelled strains and the stress state change was estimated.

The survey period at the Kylylahti mine was not sufficient for ground control risk management purposes and to create thresholds as mentioned in section 2. Longer measurement periods and multiple measurement locations are required for creating knowledge of behaviour of the support system during different mining sequences. With constant measurements threshold can be created. As said in the introduction, the creation and usage of thresholds is an iterative process. It is also important to instrument the site as early as possible to detect any changes in the early phase of the local mining sequence. The second test site at Kylylahti mine was considered to be practically useless due to late start of the monitoring and too high changes to be caused by elastic deformations. Although the risk of failure was considered to be high with the second test site, it was chosen to be monitored due the lack of alternatives. It was concluded that the modelling methods used were not sufficient enough to draw conclusion between modelled and measured strains. It was suspected that the rock mass was already highly damaged when the measurement period started.

The main results of the tests are presented in Ritala et al. (2016). The main conclusion of the results was that the method can track the changes but the results were of wrong magnitude. It was also concluded that the method is sensitive for plastic deformations. An example of the final estimated stress state changes is shown in figure 4.

![Figure 4](image-url)

*Figure 4  Example of estimated stress state change at Kylylahti mine (Ritala, 2015)*

Although cablebolts were not used in Kylylahti test site, for the purposes of ground control the usage of cablebolt extensometers is encouraged since cablebolt extensometers can also be used to give information about the loads generated to the support system. This information can be used when evaluating the
sufficiency of support, especially if the support is monitored in multiple locations. The measurement devices used and the solution for stress change are eligible for real-time risk management tools. The extensometers can be measured once per second if necessary and the stress state change algorithm can solve the stress state change from the measurement data approximately in one second. With wireless network the delay from observing the breach of threshold value to receiving the information is measured in seconds.

For ground control purposes thresholds can be created for the stress state estimation method. The first step of creating threshold values is to simulate the area of interest. The simulation method has to be based on the geological conditions. In homogenous geological conditions simple simulation methods, such as boundary element method, can be used. With complex geological conditions, more sophisticated modelling methods have to be used.

The second step of the process is to plan the instrumentation of the site. In the planning phase two important factors have to be considered. First is the geometry of the site to be instrumented. In more complex geometry more data points are required to achieve full impression of the strains and thus about the stress state changes around the area of interest. The second factor to be considered is the geology. If the area has discontinuity, such as loose interfaces between different rock domains, the areas near these points have to be monitored more comprehensively to understand their impact on stress paths.

After the instruments have been installed and the mining sequence has continued, the data received can be interpreted. After the interpretation it is vital that the data used is gathered into a database where it can be later used when the mining continues. By grouping the data into different rock mechanical zones the collected data can be used for iterative process and the knowledge of the behaviour of different rock mechanical zones are increased. The iterative process of using real-time measurements as risk management tool is illustrated in Figure 5.

![Figure 5](image)

**Figure 5** Iterative process of using real-time measurements as mine-wide risk management tool.

4.3 Predictive Analytics

Third, the Predictive Analytics is used to recognize abnormal conditions. Predictive analytics has been popularized with the advent of ‘Big Data’ and ease of data acquisition. The principle modelling technique in predictive analytics is called Similarity Based Modelling (SBM). SBM monitors all related parameters to identify patterns. From the pattern, an estimated behaviour of each parameter is generated (Gilboa et al. 2011). A central processor then compares the difference between the real-time and modelled parameter known as residual. If the residual value deviates from predetermined empirical thresholds, the system can then look at all the other related and non-related parameters that changed to evaluate what could have caused it. The advantage of such a system is that it can monitor very small residual variations and do iterative calculations to suggest possible causes. It can help gain better insight into data and find correlation
between parameters that were previously thought to be independent. The biggest disadvantage of the system is that it is data intensive and multiple parameters need to be monitored in real-time to get better estimations. However, with the increase in underground instrumentation and deepening mines, it has potential to be a tool for decision making.

5 Discussion

If rock mechanical conditions are monitored and assessed in real-time, the risk management procedure can be enhanced significantly. However, the emphasis should be put on analysis and interpretation of measured data in order to improve the understanding of interactions between rock stresses and underground excavations. The hazard potential of a particular mining area can be updated constantly and the risk assessment procedure can be initiated anytime, especially if the hazard level exceeds the pre-set limit. All mitigation measures can be monitored in real-time by continuous reliability assessment. The real-time risk management should be implemented in decision support software with user friendly graphical interface to help in the review, communication and consultation of risks.

The DIKW principle (Ackoff 1989) can be used for revision of ground support design. The understanding and connectedness of the whole process increases with the monitoring in place. In underground mines the rock stress is one of the uncertainty components in support design. Better knowledge about stress state and its changes can help to increase the design confidence level, and make better predictions about support performance in the future. Furthermore, uncertainties are also present in implementation of ground support. One of the measures to deal with uncertainty in ground support implementation is to monitor its performance and effectiveness. The latter can be done by observations and measurements, such as closure meters or extensometers. Collected data is then used for revision and optimisation of the support (Dunn, 2013).

In its best the real-time monitoring system is able to reduce the unnecessary conservativeness of the support system while increasing the safety due to monitoring and fast reaction capability. Also places with unforeseenly high seismic activity can be identified based on the increased stresses and additional reinforcement can be installed in advance.

6 Suggestions for future research

It has already been established that the continuum-discontinuum property changes around the mining stopes are too large to be ignored (Ritala et al., 2016). Geophysical non-intrusive methods are required to observe the property changes inside the rock domain. The four biggest open questions are:

- How can the excavation induced changes be transformed into rock mechanical parameters?
- How to take in account rock mass damage, plasticity and jointing during the inversion procedure?
- How to translate the measured rock mass response into geotechnical risk?
- How to carry out on-line risk assessment covering the entire mining area?

The current formulation (Kodeda et al., 2015) is linear and cannot account for rock mass damage. It may be possible to add mining sequencing and rock mass damage. If plasticity is added without sequencing, the end result may be unrealistic due to incorrect stress path. Joints are easily added given that their locations and mechanical properties are known. How to sample the rock joints and how to upscale the results is unclear, but fortunately there are already active research addressing these issues (Uotinen et al., 2015, Iakovlev et al., 2016, Sirkiä et al., 2016).

Fracture mechanics modelling can be a key to solving how the damaged zone propagates and describing how the mechanical properties of the rock mass degrade. It also connects well with the seismic predictions both with emission location prediction and with intensity studies. After the acoustic emissions counts exceed a certain threshold, the sensor array can no longer track the locations or receive the signals through
damaged rock. Fracture mechanics modelling can be used to extend the sensor arrays and simulate the observed events forward in time.

After the method development, suitable equipment selection and a review of the sites available, the system should be installed in an active mine to test its performance in in-situ conditions. This site should be fairly clean of unknown abrupt changes in geophysical and rock mechanical parameters. It is also vital that the measurements are started as early as possible to ensure that no information of changes in rock mechanical conditions is lost. The proximity to an active mine stope would be beneficial to improve the signal to noise ratio. The installation time should be before the mining passes the area of interest and the sensors should be allowed to stay long after the mining activities have stopped to detect any long term effects (e.g. creep). The selected and instrumented mine should be periodically monitored and the data continuously retrieved and saved off-site to ensure partial recovery in case of sensor loss. Based on previous experiences for Kylylahti mine, we expect to lose up to half of the sensors during the stope excavations. This is in reality an operating mine and it also provides a way to measure the minimum sensor threshold for the method solvability. Less sensors mean more noise, however this can be compensated by sampling over time.

7 Conclusion

The long-term goal is to develop a concept for real-time monitoring and risk assessment in underground mines. In this paper, we have introduced two preceding research programmes I2Mine and DynaMine and merged the concepts. The first is a classification and risk assessment system for geotechnical risks and the latter produced an elastic method for in-situ stress determination based on strain measurements. We have described real-time measurements and decision making process, reinforcement design using observational method and how to use real time stress estimation as a risk management tool. It can be concluded, that the concepts merge well together, but there are a lot of details to be described meticulously and the methods used need to be described explicitly. We have expressed our suggestions for future research. The most important research need is a successful active mine instrumentation and integration.

We have described a concept for real-time risk assessment and ground support optimization in deep mines, where stress induced problems may occur. Geotechnical hazards potential is first evaluated and geotechnical risk assessment is then defined. Real-time in-situ stress data can be used in conjunction with precalculated reinforcement and contingency actions as a basis for ground support optimisation. For the monitoring displacement measurement devices (e.g. extensometers) are best suited from elastic to cracking rock and acoustic emissions or seismicity measurements for damaged rock calculations. The proposed method works best when installed close to mining activities at sufficiently early state to pick up the stress changes induced by the mining process.

Acknowledgement

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Remediation of large scale structures in open pit mining

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Abstract

The mining of iron ore in LKAB’s open pit mine at Gruvberget, Svappavaara has been associated with rock mechanical issues from the start. A rockslide in 2013 called attention to a large structure or shear zone in the southwestern corner of the pit. This structure has presented problems on every new bench since then. Different methods have been used to remedy the effects of the structure on workplace safety and stability of the pit wall. These methods include: (i) early attempts of stabilizing the rock with rockbolts and mesh, (ii) reshaping the bench and slope geometry, and (iii) stabilizing the access ramp with a waste rock buttress. These measures have permitted the continued mining of the deposit close to the original plan. This paper describes the different remedial measures employed, together with the practical experiences and lessons learnt.

1 Introduction

LKAB (Loussavaara-Kirunavaara AB) is a government-owned Swedish mining company that specializes in iron ore mining and production of pellets for blast- and direct reduction furnaces (DR). The main supply of iron ore for the company comes from two underground mines situated in the cities of Kiruna and Gällivare. An overview of the company’s operations in Sweden and Norway can be seen in Figure 1 below.

To increase production and lower the operating cost, LKAB is in the process of starting up and expanding open pit operations in and around the town of Svappavaara, situated between Kiruna and Gällivare. This paper will focus on the remediation efforts and lessons learned during operations of the pit Gruvberget. The pit and its surroundings can be seen in Figure 2 below.

Gruvberget is the smallest of the planned open pit operations in the Svappavaara field, but at the time of writing is the only one in full production. The main product is a high-grade magnetite iron ore that is used directly in the on-site pelletizing plant. Since the start in 2010, the annual production has been 2.0 Mton of ore through the crushers, with a strip ratio of approx. 3.5 over the lifetime of the pit. The planned pit for stage 1 and 2 of the project will be roughly circular in shape with outside dimensions of 600 x 600 m. The final pit slope will be roughly 270 m high in the southern part of the pit. The pit is situated on the side of a mountain, Gruvberget (from which the pit is named), which gives a height difference of about 100 m between the north and south slope. The pit is constrained on all sides by objects that require special attention and/or permit applications before mining: Directly to the north and west of the pit is the lake Syväjärvi, and the tailings dams and water treatment ponds for the dressing and pelletizing plants. The lake is also, in addition to its water protection status, a popular recreation area for the locals. To the east, about 500 m away, lies the main highway to the north of Sweden (E10) and the railroad and terminal for the import/export of goods to and from the plant. In the south, there are remains of the earlier mining history of Svappavaara: Old pits and building foundations from the 17th through the 19th century that are listed as protected objects by law. Mining is done in 15 m benches, double-stacked with a catchment bench every 30 m. Bench face angles are between 80° and 60°. Final pit slopes are pre-split were applicable.
Figure 1  Overview of LKAB operations in northern Sweden and Norway

Figure 2  Overview of LKAB mining operations in Svappavaara
2 Case history: Gruvberget, southwest pit walls and ramp

2.1 Early attempts (pre-2013)

During the initial stages of mining there were issues with rock stability, primarily on the western side of the pit. Before the fall of 2012, LKAB had no in-house rock mechanical personnel with experience from open pit operations. The consultants hired recommended using chain link mesh with fully grouted rockbolts as the primary reinforcement method for large parts of the western wall and other areas with unstable rock. The method was not efficient: within a year of installation, a large amount of the bolts were only embedded 0.5-1.0 m in the rock due to spalling of the rock around the bolts. The rock on the western side consists mainly of biotite schist, highly jointed with low strength. An example of this can be seen in Figure 3 below. In addition, despite a considerable amount of loose rock behind the mesh, the spalling showed no sign of abating. The method also called for a large volume of reinforcement installations in the future. This changed from 2012 to focus more on mechanical scaling of all final pit walls. Mesh was reserved for the areas with the worst/softest rock conditions to prevent the loss of catchment benches due to overscaling.

![Figure 3](image3.jpg)

**Figure 3** Rockbolts protruding out of bench face 0.5-1.5 m after spalling (red markers). The anchor bolts for the mesh were initially installed 3-5 m behind bench crest. In this image, they are situated on the bench crest due to continued spalling/backbreak. The debris screen formed over 2-3 months of continued spalling, the area was initially pre-split and scaled.
2.2 Rockslide of April 2013

2.2.1 Failure description
A large wedge failure occurred in late April of 2013 in the southwestern part of the mine when the lower part of the bench between levels 370 and 340 was being mucked out. Fortunately, signs of the collapse were evident a few hours before and the area was evacuated. The failure was estimated to be in excess of 10,000 tons. The full extent of the failure can be seen in Figure 4 below. The primary causes of the failure were the formation of a wedge between a large scale structure in the south wall, the foliation planes occurring in the schistose rock in the western wall and the planned slope direction that then collapsed when the supporting rock at the toe was excavated. The foliation planes were well known, believed to be the cause of much of the earlier rock stability issues on the western pit walls. The large-scale structure was however previously unknown, as this was the first time the pit had extended this far west. The structure is classified as a 2-4 m wide shear zone with a mixture of crushed materials and heavy schist alteration. No direct continuation of the structure can be found in the opposing pit walls.

![Figure 4 Site of rockslide the day after the event, looking from northeast to southwest. Note the steeply dipping structure on the left side, forming an overhang and the smooth foliation plan on the right.](image)

2.2.2 Remediation of failed area
The immediate work focused on securing the structure and the work site surrounding it. The dip of the structure left a sizeable overhang on the eastern side of the failure area, which hindered all clearing and scaling operations. A radio controlled drill rig was hired to facilitate drilling and blasting of the overhang. Then, using the broken rock as an access ramp, the failure area was scaled using a long reach excavator with a hydraulic hammer. This remediation effort had the effect of hindering waste rock mining in the area for a period of 1-2 months.

2.2.3 New pit geometry
During the remediation, a new layout of the pit was put forward that intended to address most of the issues encountered. The main changes proposed was a lowering of the bench face angles below the average dip of the foliation in an attempt to undercut these and stop further unravelling. To reduce or eliminate potential future wedge failures, the slope direction for the south wall was altered and the corner...
between the west and south wall was made less rounded. The results of the new layout can be seen in Figure 5 and Figure 6 below.

Figure 5  New pit layout on the left overlayed with the old pit layout on the right. Note the straightening of the pit wall on the western side, leading to increased waste rock mining, and the lowering of the bench face angle, leading to ore loss in the bottom of the pit

Figure 6  Slope geometry for west and east side of pit, purple line is old layout

The new layout, however, led to increased waste rock mining which in turned delayed ore production later in the year. The changes to the slope direction were also only possible to achieve in the top 90 meters before constraints closer to the pit bottom rendered them impossible.

2.2.4  Could this have been avoided?

The slope design at the time of the failure was based on a design-as-you-go approach, partially because of personnel shortage in the early stage of the project. Some GSI-mapping of earlier bench faces and surface structure mapping had already led to a lower bench face angle for the western face than the rest of the pit. It is possible that a more in-depth study (structure mapping in drill holes/tele-viewers) could have identified the structure that was part of the failure, but a design of the bench face angle on the western wall would probably have led to the same conclusion based on structure orientation alone.

The pit boundaries in themselves were also an issue in any layout change that was to occur: The southern and western slope crest are almost at the edge of the mining permits already, due in part to the occurrence
of the listed historical remains, but also in a restrictive approach to permit application in the early project stages.

2.2.5 Subsequent issues

From the 340 level down, each new bench crest and face that intersected the structure needed heavy scaling. This resulted in the loss of about half the catch bench width on all benches for a length of 10-30 m. A gradual decrease in the dip of the structure could also be observed. This meant that the overhanging rock on each consecutive bench grew larger. To counter the loss of catch bench area, mesh was used to slow falling rocks down and reduce their energy. The installation of mesh was in turn difficult because of the loss of bench width that hindered access to areas and forced the installation of anchor bolts further apart. The rock fall hazard was also present, as shown in Figure 7 below.

![Figure 7](image)

Figure 7 Rock fall while installing anchors. The rock fall occurred after heavy rainfall, when no personal was present in the catch bench area.

Due to the extended application of mechanical scaling close to the structure, no uncontrolled failures of the rock occurred. A new structure similar to the first, was discovered further west with the pushback created by the new layout. This structure also caused issues with overbreak from scaling and overhangs, similar to the first, but had a different dip and strike which caused it to disappear after only two benches.

2.3 Access ramp failure in 2015

2.3.1 Failure description

Due to the small size and low production in the mine, all access is via a single, dual lane haul ramp. The designed ramp width was 26 m for an effective road width of 18 m after considerations for backbreak, safety berm and ditch had been taken. On the first bench were the ramp intersected the structure, the crest was relatively unaffected. When the lower bench was mined, the structure was found to undercut more than expected. A number of scaling operations was performed on the overhang during early 2015 (February - April) which led to an effective road width of only 10-12 m. An example of this can be seen in Figure 8 below. This meant only single lane traffic was possible for a length of approximately 50 m of the haul road.
Rock Slope Stabilization

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Figure 8  First scaling of the structure below main ramp in February 2015. Previous intersections on bench faces by the structure arame marked with a red line

After a large ingress of surface water (due to spring thaw), which washed the fine material out of the remaining structure below the ramp and destabilized the bench face further, management closed the ramp for heavy traffic citing concerns for further stability issues. There was also concern for the psychological welfare of workers, forced to drive with the cab-side of the trucks close to the edge on upwards trams. To resume production, a number of emergency measures were instituted:

- Extensive scaling and mucking debris from the affected area. This led to an even longer stretch of haul road with single lane traffic. The effects of this can be seen in Figure 9 and Figure 10 below.

- Surface water containment: the haul road ditch was led past the area in pipes. This had the benefit of increasing the road width with 1-2 m due to the backfill over the pipes.

- Scaling and cleaning of the inside bench face: This added another 1-2 m width to the road, but removed some of the catchment bench above.

- Construction of a temporary single-lane ramp with waste rock material from above the affected area to the pit floor as seen in Figure 11 below. The vertical distance to the pit floor was only 35 m at that moment.

- Traffic restrictions for vehicle, making the ramp one-way: Only unloaded trucks passed the failed area and loaded trucks used the temporary ramp.
Figure 9  Structure after final scaling campaign, May 2015. The result of a previous scaling of the structure above the haul ramp can be seen on the right.

Figure 10  Loss of effective haul ramp width after scaling. An estimated 6-8 m of extra backbreak can be seen.
2.3.2 Plan of remediation

During the time that the above-mentioned measures were put in effect, a proposal was formulated to deal with the effects of the ramp failure. The goal of the proposal was to resume normal mining with as little effect on ore reserves and disturbances to production. The proposal had three main options:

1. Resume two-way traffic (passing traffic) past the failed area without further measures than the ones described in the previous chapter.
2. Keep the temporary ramp with continued one-way traffic system.
3. A support- or bridge construction past the failed area. After consultations, a bridge construction was deemed unfeasible due to difficulties in securing footing for the construction. The support options was narrowed down to two alternatives, both designed to stop further failures and stability issues:
   a. Concrete retaining wall and buttress construction
   b. Rock fill buttress.

These options had their respective strength and weaknesses, as summarised in table 1 below.

After consideration, option 3b was selected as the best alternative for the situation considering costs and the uncertainties in the other proposals.
### Table 1  Strengths and weaknesses of proposed options

<table>
<thead>
<tr>
<th>Option</th>
<th>Strength</th>
<th>Weakness</th>
</tr>
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<tbody>
<tr>
<td>1. Resume two-way traffic, one-lane, no further support.</td>
<td>No major change to mine layout, therefore minimal ore loss.</td>
<td>Uncertainties about stability, heavily discouraged by management for health &amp; safety reasons.</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Production loss from queuing at chokepoints/bottlenecks (two-way, one lane traffic).</td>
</tr>
<tr>
<td>2. Two ramps, one-way traffic.</td>
<td>Added redundancy with a second ramp against future failures.</td>
<td>Pit outline is fixed, a second ramp would lead to ore loss (approx. 1.0 Mton).</td>
</tr>
<tr>
<td></td>
<td>Traffic benefits from two lanes, one-way.</td>
<td>Crossing/left turn heavy traffic at ramp intersection.</td>
</tr>
<tr>
<td>3a. Concrete retaining wall and buttresses.</td>
<td>No major change to mine layout, therefore minimal ore loss.</td>
<td>Construction time: 4-6 months (detailed design and construction).</td>
</tr>
<tr>
<td></td>
<td>Regain width for two-lane, two-way traffic.</td>
<td>Construction cost &gt;10 MSEK.</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Ramp unavailable during construction.</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Additional contractor needed for concrete construction.</td>
</tr>
<tr>
<td>3b. Rock fill buttress.</td>
<td>Construction time: 1-2 months.</td>
<td>Ore loss (approx. 0.3-0.4 Mton) due to space requirements.</td>
</tr>
<tr>
<td></td>
<td>Construction could be handled with existing mining contractors/in-house resources.</td>
<td>Production loss from queuing at chokepoints/bottlenecks (two-way, one lane traffic).</td>
</tr>
<tr>
<td></td>
<td>Ramp available during construction.</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Lower cost for waste rock handling (temporary, due to shorter haul distance).</td>
<td></td>
</tr>
</tbody>
</table>

### 2.3.3  Design, construction and final result

The buttress was designed to support the rock behind it and the additional load of a fully loaded mine truck (approximately 200 tons). To be stable in itself the slope of the buttress was calculated to 1:1.5. A sketch of the planned buttress can be seen in Figure 12 below.
Construction of the buttress started in July 2015. Waste rock was taken directly from the mining operation in the pit, leading to short haul distances. The buttress was constructed in layers of 2 m thickness, with every layer compacted by a vibrating roller. For workplace safety, the face was scaled at regular intervals. To minimize erosion from surface water the surface of the buttress was compacted and smoothed using an excavator. The western side of the buttress was constructed with a lower angle and slightly wider to facilitate access for future repairs. This is located in the corner of the catchment berm and does not affect the layout further. A berm and a catchment bench of approximately 4 m width were left in front of the buttress. The final result can be seen in Figure 13 below.
2.4 Continuing efforts

Since construction of the buttress, the first bench of the next level below has been mined. The structure was evident here also, but to a lesser degree. Some extra scaling of overhangs was required, but not enough to endanger the stability of the buttress. The mining of the lower bench is about to be completed in early 2016.

The rock mechanical problems continue in other parts of the pit as well, mostly in conjunction with the schistose rock on the western wall. The spalling/slabbing of the schist continues to a lesser degree than before, but still requires ongoing scaling efforts every now and then. Since the pit has passed the groundwater level in the last few years, issues with water ingress (mainly production-related) has increased.

3 Conclusions

- Mesh and bolts were not enough to stop the unravelling of bench faces, they only served to slow down rock falls to increase the effectiveness of the catchment benches.
- The large wedge failure could probably have been avoided if a more thorough slope stability studies had been performed earlier in the project. The initial slope design for the western wall however, would likely remain unchanged based on the structure mapping.
- It is possible to handle unforeseen large-scale structures within day-to-day operations; it will however require more flexibility in the mine organization as a whole (geology, rock mechanics, planning, management and operations). In the case of Gruvberget, this was alleviated by the small size of the organization.
- Using a waste rock buttress was the most economical option concerning time, cost and ore loss that also fulfilled the safety requirements. The buttress also enables the mining of future pushbacks with relative ease.

Acknowledgments:

The author would like to acknowledge the following companies and persons for their support during the mining process: Pöyry AB for the initial rock mechanical consultation work, NCC Consulting & Construction AB for early estimates of construction options for the access ramp and final design of the buttress and Itasca Consultants AB with Dr. Jonny Sjöberg for rock mechanical consultations and second opinion work during the length of the project.
Rock bolt behaviour in stress-fractured ground - A FE analysis using zero-thickness interface elements and fracture-based constitutive laws

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E. Eberhardt, University of British Columbia, Vancouver, Canada

Abstract

A finite element analysis using zero-thickness interface elements of the Goodman type, which incorporate a fracture mechanics-based mixed-mode constitutive law, is presented to study stress-induced cracking and spalling around a highly stressed tunnel. The effect of bolting on stress-fracturing and rock mass deformations (strains of the rock matrix and opening/sliding of the fractures) is considered by applying bolts at various spacing densities. The adopted tunnel excavation and far-field loading sequence approximates conditions typically experienced at the extraction level of a caving operation. Starting from an in situ stress state, the tunnel is excavated and then loaded vertically to simulate the advancement of the overlying undercut. The tunnel is then unloaded to stress values lower than in-situ once the undercut becomes effective and caving begins.

Using a Voronoi tessellation routine, the rock mass near the tunnel cross-section is randomly discretized into triangular elements of similar size to that expected for spalled rock blocks. Interface elements are inserted in between all mesh lines. An interface transitioning to the far-field rock mass separates this finely-discretized zone allowing the rest of the cross-section to be modelled as a linear elastic material for computational efficiency. Rock bolts are simulated in a simplified manner with elasto-plastic rod elements overlapped to the continuum mesh and anchored at several points along their length including the ends. Calculations are run using various bolt spacing densities. The model results show bulking due to stress-fracturing of strong rock. They further show that bulking can only be reduced, but not prevented, through confinement provided by high bolting forces.

1 Introduction

Numerical analysis of tunnel stability has traditionally been approached in two ways: as a continuum and as a discontinuous (discrete) problem. In the first approach, Finite Difference Method (FD), Finite Element Method (FEM) or Boundary Element Method (BEM) formulations are equipped with elastoplastic or damage laws with softening to represent cracking (Hormazabal 2010). However, such representations are intrinsically limited by the continuum assumption itself, since it is not possible for instance to represent the opening or sliding of a single crack plane without introducing deformations at the same time in other neighbouring planes. The second approach consists of methods that consider the rock mass as a number of blocks separated by contact surfaces. This group includes methods such as Distinct Element Method (DEM), meshless methods (such as Material Point Method, MPM) and hybrid FEM/DEM techniques (Vyazmensky et al. 2010; Elmo et al. 2013). In addition to a more realistic kinematics of highly stressed brittle rock failure, some of these methods may also simulate the processes of block (element) fragmentation (transition from continuum to discontinuum). The latter requires significant computational power and may be conditioned by factors such as the initial layout of the mesh, or the constitutive model used for the emerging cracks/discontinuities.
In this paper, a discontinuous approach based on FEM with zero-thickness interface elements is adopted for the analysis of quasi-brittle rock materials. In particular, the paper focuses on the analysis of pillar stability between adjacent underground excavations (tunnels) representative of the extraction level of a block caving mine (Fig. 1).

![Figure 1](image1.png)

**Figure 1** Block caving: a) scheme of a caving mine (mechanised panel caving, Henderson Mine, Colorado, USA; Doepken 1982); b) plan view of a herringbone extraction level layout (Kvapil, 2004; taken from Pierce, 2010)

2 FE model for brittle rock based on zero-thickness interface elements and a fracture energy-based constitutive law

The cracking schemes observed near deep tunnels in brittle rock, typically include two types of cracks: (1) inclined shear-compression fractures (often at 30-45° to the tangential loading direction), and (2) near vertical extensile opening cracks (parallel to loading direction), as shown in Fig. 2.

![Figure 2](image2.png)

**Figure 2** Stress-fractured ground near excavation showing both shear (reverse arrows) and extensional movements (open arrows). From Kaiser and Cai (2013)

In this paper, all types of cracks, including the two types discussed above, are represented by joint/interface elements of the zero-thickness “Goodman” type (Goodman et al. 1968), which are equipped with the appropriate fracture-based energy-driven constitutive law.
2.1 Zero-thickness interface elements and geometric layout

Interface or joint elements of zero-thickness type (Goodman et al. 1968) are a special kind of finite element introduced between adjacent elements. Their singular feature is that they have one less dimension than the standard continuum elements; i.e., lines in 2D and surfaces in 3D. The integration of these elements is done through a local orthogonal coordinate system defined on the interface line or surface.

Due to the reduction of dimensions, the interface constitutive behaviour is formulated in terms of the jump of the main variable across the mid-plane of the interface, and the corresponding force-type conjugate variable. In the standard mechanical problem, these variables are the normal and tangential components of the relative displacements (Fig. 3), together with their counterpart stress tractions on the interface plane.

Figure 3 Zero-thickness interface elements and associated kinematic variables (relative displacements)

Although zero-thickness interface elements were initially proposed to represent pre-existing discontinuities or material interfaces of known fixed location and frictional behaviour (Goodman et al. 1968, Gens et al. 1988), their application was later extended to represent newly developing cracks of an initially intact media (e.g., Rots 1988; López and Carol 1998). In this context, the approach consists of inserting interface elements along all (or a subset of all) mesh lines, and equipping them with a fracture-based constitutive law with the initial strength of the uncracked material (Garolera et al. 2011). The cracks will then open or close depending on the stress distribution and boundary conditions. In this approach the mesh layout may be important, since it must include the main potential crack paths as mesh lines. The procedure to generate such random unstructured meshes is based on Voronoi tessellation plus further subdivision as shown in Fig. 4a, leading to a mesh such as the one in Fig. 4b.

Figure 4 (a) Voronoi Polygons and further subdivision, and (b) final mesh with interface elements
Note that in the final mesh with interface elements (Fig. 4b), each original node is superimposed with as many nodes as mesh lines that reach that point, and therefore, the mesh size increases considerably. For this reason, this subdivision is only used in critical areas of the mesh where cracking is expected to occur. The rest of the simulated domain is discretized as an ordinary continuum (Section 3.1).

2.2 Mixed-mode constitutive mode for interface elements based on fracture energy

The constitutive mode used for opening and sliding of the zero-thickness interface elements is the one proposed by Carol et al. (1997), which was later improved by Caballero et al. (2008). The model is based on a hyperbolic loading surface (Fig. 5a) expressed in terms of normal and shear stress (respectively $\sigma_N$ and $\sigma_T$) on the interface plane:

$$F = \sigma_T^2 - (c - \sigma_N \tan \phi)^2 + (c - \chi \tan \phi)^2$$  \hspace{1cm} (1)

The three parameters of the hyperbola, i.e., tensile strength ($\chi$), apparent cohesion ($c$) and friction angle ($\phi$), have initial values determining the initial shape of the cracking surface (Fig. 5a). The surface may be reached at different points representing different fracture modes. Pure tension (Mode I) and shear-compression (Mode IIa) represent two limit situations (Fig. 5b) with characteristic fracture energies $G_{fI}$ and $G_{fIIa}$, while intermediate points represent mixed-mode cracking. Once the incipient crack starts opening and/or sliding, the surface parameters start evolving (Fig. 5c). Their evolution is governed by a history variable $W^{cr}$, work spent in fracture processes per unit area surface of developing crack, that reflects the energy spent in a fracture per unit area of interface (Fig. 5d and 5e).

![Figure 5](image)

**Figure 5** Constitutive model of the interface elements: (a) fracture surface and plastic potential; (b) two fundamental modes of fracture; (c) evolution of the fracture surface, from initial state “0” to exhaustion of tensile capacity “1”, to residual state “2”; (d) and (e) softening laws including various shape parameters for better fitting of experimental data (Carol et al. 2001)

The increments of $W^{cr}$ are assumed to equal the increments of fracture work in tension or shear-tension, and to the shear work minus frictional dissipation in shear-compression:
In this expression, \( u_{N \text{cr}} \) and \( u_{T \text{cr}} \) represent the opening and sliding of the interface, that in the context of elasto-plasticity are defined as the total relative displacements minus the stress-related elastic ones, i.e. 
\[
 u_{N \text{cr}} = u_N - u_{N \text{el}}, \quad u_{N \text{el}} = \frac{\sigma_N}{K_N}
\]
for the normal component, and similar equations for the shear component. The flow rule is assumed to be associated in tension or shear-tension, and non-associated in shear-compression. Thus, the dilatancy predicted by the associated flow rule is reduced by two factors reflecting compression on the cracking plane and the extent of degradation already suffered by the interface.

2.3 Verification of failure pattern for specimens under different loading conditions

The approach described has been implemented in the FE code “DRAC”, developed in-house by the MECMAT group of the Division of Geotechnical Engineering of the Civil Engineering School at UPC. The approach has been verified extensively for failure of concrete and rock, in a variety of loading situations in 2D and 3D (Gens et al. 1995, Caballero et al. 2007). Before performing the tunnel analyses, simulations of laboratory testing of synthetic rock specimens (discretized using the same techniques as in Fig. 4b) were used to calibrate the interface parameters so that reasonable values are obtained for overall behaviour in uniaxial tension and compression, as well as triaxial compression and extension.

3 Application to the tunnel loading scenarios

3.1 Geometry

The “reference mesh” of the tunnel cross-section modelled is depicted in Fig. 6. Due to symmetry, only half of the tunnel is represented. The mesh includes a zone of interest in which interface elements have been inserted along all mesh lines, and a far-field zone assigned standard elastic continuum properties. The two zones are separated by a transition interface. Values for the elastic parameters of the interfaces are set to very high values such that the overall elastic behaviour of the denser mesh is not altered substantially by their presence. The interior of the tunnel is initially discretized to be able to represent the excavation process as a first step. Five horizontal bolts are included in the tunnel wall but are not activated until a later stage in the simulation. The modelling of the bolts is implemented via one-dimensional two-node bars, which are anchored to the surrounding rock elements at various points including the two end nodes, and in most cases, three intermediate points. This model represents an array of tunnels separated by pillars with a large width/height (W/H) ratio such that the tunnels do not affect each other.
3.2 Loading and boundary conditions

The geometric and loading conditions considered for the calculations are:

**Step 1: Initial stress state** - Full cross-section before excavation of the tunnel and installation of the bolts, subject to in-situ stress state with values $\sigma_v = 30$ MPa and $\sigma_h = 45$ Mpa, consequently with horizontal-to-vertical stress ratio ($K_0$) equal to 1.5.

**Step 2: Excavation of the tunnel cross-section** by application of the stress traction on the tunnel cross-section perimeter while maintaining the vertical stress on the top boundary of the domain, and zero normal displacements on the remaining three external boundaries.

**Step 3: Placement of bolts, and increase of vertical stress** - Five horizontal steel bolts are installed at the tunnel wall. The calculations of this and later steps are repeated for various assumptions regarding the bolts: i) no bolts, ii) bolts anchored only at the two ends, iii) bolts anchored at several points, and iv) bolts installed with three different spacing densities. For each of the bolt scenarios, the vertical stress is then increased at the top boundary of the domain from 30 to 60 Mpa to simulate the abutment stress of the advancing overlying undercut.

**Step 4: Decrease of vertical stress** - For each of the bolt scenarios, the vertical stress is decreased from 60 to 15 Mpa to simulate the completion of undercutting and beginning of mining.

**Steps 5 and 6: Additional decrease/increase of vertical stress** consisting of further decreases of distributed load on the upper side of the domain from 15 to 5 MPa followed by increases from 5 to 15 MPa to simulate variable stresses during ore extraction and mine operations.

3.3 Results

Figures 7 to 12 illustrate the results obtained for three different bolt load capacities of 8.4, 25.0 and 50.4 t (tonnes) at a tensile strength of 600 MPa. The 50.4 t bolt, in practice, would represent multiple bolts rather than a single bolt. The bolts are assumed to have an infinite yield strain capacity. In other words, they are allowed to yield but do not fail in these simulations.

Fig. 7 presents the results obtained for the case without bolts, at the end of Step 3, after increasing the remote vertical stress from 30 to 60 MPa. The largest displacement values obtained in this case are horizontal displacements of 25.0 cm at the tunnel wall (with vertical upward displacements of 7.9 cm at the tunnel floor, and -11.0 cm downward at the roof). The fracture patterns depict deeper shear-like cracks
forming wedge-like blocks (inclined about 45 degrees from the corners into the ceiling and floor, as well as into the tunnel wall), combined with tension-like vertical cracks near the tunnel wall. Despite the applied $K_0 = 1.5$, stress-damage occurs around the entire tunnel but more energy is consumed in the walls, and spalling by tension is essentially constrained to the walls. More bulking occurs in the walls than in the roof or floor.

When the remote load is decreased from 60 to 15 MPa (Step 4) and to 5 MPa (Step 5), and then increased again to 15 MPa (Step 6), the number of interfaces that overtake the strength criterion increase rapidly producing widespread cracking and damage. The loading/unloading states for Stages 4, 5 and 6 without bolting are shown in Fig. 8. Note that during unloading the damage reaches the interface transition to the elastic rock domain. Fracturing in Fig. 8b and c is therefore limited by the elastic rock domain. Interestingly, the fracture energy spent does not seem to change much for this model. The deformations keep increasing during unloading, however only by small amounts (largely due to the elastic domain control).

![Figure 7 Results for the case without bolts at the end of Step 3 (after increase of vertical stress from 30 to 60 MPa): (a) total displacement in cm; (b) energy consumed at the interfaces normalized to fracture energy for mode I, $G_f$, and (c) interface state (loading or unloading) for all interfaces that exceed the strength criterion](image-url)
The effect of using bolts with increasingly higher yield capacity is presented in Figs. 9 and 10. The corresponding axial stresses in the bolts are shown in Fig. 11 and the respective yield patterns are presented in Fig. 12. The bolts practically eliminate the near surface parallel (vertical) tensile cracking in the walls. As a consequence, the tunnel wall displacements reduce (to about half). The bolts, however, have a minor effect on the loading/unloading cracking state (not shown) that remains similar to the state shown in Fig. 8.

The models with bolts included also allow the stress and deformation state of the bolts themselves to be calculated. For the results presented here, each bolt is represented by four rods (or segments of the bolt) anchored to five nodes belonging to the background rock. Within each of these four segments, the bolt exhibits constant stress and deformation. Given the significant displacements of the tunnel wall (over 15 cm even for the 50 t bolts), and considering the bolts elasto-plastic behaviour, the results show that after Step 3 all bolts have exceeded the tensile strength limit in at least one of their segments (e.g., Fig. 11; over the entire length in the bolt at mid height). The accumulated plastic strain exceeds the yield capacity of steel near the tunnel wall, especially for the mid to lower bolts (Fig. 12). Thus, these bolts would have failed, which is not captured in the simulations.

After unloading from 60 to 5 MPa (Step 5) and reloading to 15 MPa (Step 6), some of the bolt segments deeper in the rock become unloaded, although closer to the tunnel wall stress remains at the rock’s maximum tensile strength and the plastic deformations keep increasing in many parts of the model.
Figure 9  Displacement field obtained at the end of Step 6, for elasto-plastic bolts of various yield capacities (displacements in cm)

Figure 10 Crack damage map in terms of fracture energy spent at the end of Step 6 for elasto-plastic bolts of various yield capacities
Figure 11  Stress state in bolts after Step 3 (remote stress to 60 MPa) for various bolt yield capacities. Stress units in MPa

Figure 12  Strain state in bolts after Step 3 (remote stress to 60 MPa) for various bolt yield capacities. Stress units in MPa

Fig. 13 presents a curve similar to a Ground Response Curve (GRC). The data points are obtained from the calculated bolt forces and displacements, and are referenced to the timing of the bolt installation (i.e., any minor deformations accumulated before bolt installation are neglected). The sum of the bolt forces at their contact point with the tunnel wall, divided by the height of the tunnel wall, is used to calculate the average support pressure exerted by the bolts on the tunnel wall. Each pair of values (average pressure and wall
displacement) provide a point on the GRC curve, which can be compared to the support reaction lines for each bolt capacity scenario.

The bolt capacity scenarios analysed in the previous figures (i.e., 8.4, 25 and 50 t capacity) can be seen to provide three different intersection points with the GRC curve, all in the lower range of sustaining pressures under 1 MPa. This does not allow a proper characterization of this curve. For this reason, additional calculations with higher bolt capacities of 118.8, 221.1, 475.0 and 646.5 t were also run. Although these yield capacities are not realistic, the results provide the additional points required to plot the full GRC, because the end stage of each calculation provides an average bolt pressure and corresponding tunnel wall displacement for the given bolt capacity. As the tunnel wall displacements are not uniform, the average displacement of the five bolt ends is taken as “wall displacement” for this purpose.

As expected, for hypothetically higher bolt capacities, the calculation progressively gives lower wall displacements and cracking associated with more limited yielding of the bolts. For hypothetical bolts with 646.5 t capacity the calculations show that such bolts would remain elastic with an average sustaining pressure near 5.5 MPa. At this stage, cracking and wall displacements is practically suppressed.

4 Concluding remarks

The results obtained show a clear representation of brittle fracture damage and dilation zone development, with two key mechanisms at work: (i) deep shear-compression cracks inclined at orientations near 60° forming larger irregular blocks, which in turn contain (ii) shallower vertical opening extensile cracks, parallel and closer to the tunnel walls. The displacements obtained at the tunnel walls are in the order of 10 to 20 cm (due to rock mass bulking), and are reduced progressively with higher bolt densities (increased confinement). Additional useful information that can be extracted from the numerical results includes, among others, the depth of rock affected by cracking and decompression (i.e., depth of damage).
Whereas some of bolt capacities used in the analyses are far beyond practical conditions (average support pressures typically range between 0 and <1 MPa), the results show that stress-fracturing and related bulking cannot be prevented by bolting, but can be reduced. Such bolts must be designed to resist the large strains resulting from block separation and rock mass bulking. For the model results presented here, average bolt strain higher than 5% with locally much higher strains (>20%) were observed. This supports the practical observations of strain localization leading to bolt failure in stress-fractured ground. Ongoing work aims at including the effect of shotcrete lining in the analysis and at extending the approach to 3D.

Acknowledgement

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References


Scale effect of thin spray-on liners for pillar reinforcement

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Abstract

Thin spray-on liners (TSLs) are being used for rock reinforcement support in underground mines. They have wide ranging areas of application from shotcrete rehabilitation to pillar reinforcement systems. Pillar support mechanism investigation is a common study subject among TSL researchers. TSL coated rock samples are loaded uniaxially and compared with uncoated test results, while acknowledging that small scale tests may not represent the in-situ conditions.

In this study, reinforcement capacity of TSLs are tested with coated rock samples having different diameters. A cement based TSL consisting of mineral additives and copolymer is used in the laboratory to coat the homogenous andesite cores to mimic pillar reinforcement effect. The uniaxial compressive strength (UCS) test failure loads of uncoated and 5 mm coated cores having 25, 55, 75, and 100 mm diameters are compared. The TSL’s material properties are also determined with tensile and deformability tests based on the related ASTM standards (D-638, D-695). The results of coated core compression tests indicate that there is no significant strength improvement for the homogenous rock cores coated with less than 20% (TSL thickness to core diameter ratio) coverage. Therefore, it is concluded that TSL pillar confinement effect should be studied with post-peak behaviour of the tests.

Keywords: Thin spray-on liner, pillar, confinement, UCS, coating.

1 Introduction

Shotcrete is one of the primary means of support in underground excavations since the invention of New Austrian Tunneling Method (NATM), starting in early 1970’s (Mirzamani et al. 2011). During the rapid technological advancements in the mining industry, improvement in mechanical properties of shotcrete became necessary. In the late 1980’s, MIROC (Canadian Mining Industry Research Organization) initiated detailed research with the support of private companies, in order to develop an alternative material to shotcrete. As a result of this research, they improved the first thin spray-on liner (TSL), a polyurethane-based product. General definition of TSL is” generally cement, latex, polymer-based and also reactive or non-reactive, multi-component materials applied to the rock surface sprayed by nozzle, in a layer of generally 6 mm or less (3-5mm) thickness material”.

Although, TSL’s invention about 30 years ago was because of its benefits on improvement on logistics, cycle times, mechanization and safety, many TSLs have been developed and modified to answer the need of the mine support systems. These thin, rapid setting, polymeric/polyurea or cement based coating materials are now being considered for highly stressed, deep, rock bursting conditions as a reinforcement and retaining element together with bolts and screen (Swan et al. 2012).

TSL products are predicted to be effective support systems by TSL researchers; however, mechanical behaviour of these surface support materials are still questionable, due to lack of understanding of the supporting mechanism (Lau et al. 2008). Surface support mechanisms of TSL’s have been described and summarised by Stacey (2001) and Tannant (2001). Researchers have studied to understand the support mechanism of TSLS using numerical, analytical methods and laboratory studies (Lau et al., 2008; Ozturk, 2012). In the literature, 13 different laboratory test setups were utilised (Guner and Ozturk 2016) and among them, coated core compression test is the most preferred one. The major purpose of coated core test is to simulate the confinement that TSLs can have on pillars. This test method was first proposed by Espley et al. (1999). After this study, various researchers performed the same test to understand the pillar
support mechanism of TSLs (Archibald and Degange 2000, Kuijpers 2001, Tarr et al. 2006, Lau et al. 2008, Gilbert et al. 2010, Han et al. 2014, Qiao et al. 2014, Li et al. 2015). The most extensive study was performed by Qiao et al., (2014). Specimen preparation, loading rate and the mechanical properties of the rock and the liner were described in detail. Three types of rock, with and without a liner, were loaded by uniaxial compression and significant strength improvements were observed in the coated samples especially for weaker rock samples.

In the literature, researchers generally focused on the enhancement of the uniaxial peak failure values of the coated specimens. Usually, 55mm cylindrical specimens coated with 5mm thick liner and cured for 7 days were tested. Strength enhancement was observed in almost all the studies. However, this enhancement may not be representative for the in-situ behaviour experienced in underground excavations, due to high pillar widths/diameters compared to limited liner thickness covered in practice.

In this study, the effect of specimen size on the support mechanism of TSLs for pillar reinforcement is investigated. For this purpose, different sized core specimens were coated and tested.

2 Laboratory studies

2.1 Tested Liner

The laboratory tests are conducted with Tekflex LP (Minova Carbo Tech) liner. The liner has two components; the liquid-component is a stabilised resin latex and the powder-component is a hydraulically curing powder based on special cement, packaged in 20kg bags. Components are mixed with 2:1 liquid-powder ratio. Composition of the liner is not shared by the manufacturer.

2.2 Sample preparation

Homogenous and intact andesite core samples are used in this study. The mechanical properties of andesite samples are presented in Table 1. Note that the deformability tests are performed with 75 mm diameter cylindrical cores.

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
<th>Std. Dev.</th>
</tr>
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<tbody>
<tr>
<td>Uniaxial Compressive Strength (MPa)</td>
<td>66.9</td>
<td>5.10</td>
</tr>
<tr>
<td>Tensile Strength (MPa)</td>
<td>8.59</td>
<td>0.32</td>
</tr>
<tr>
<td>Young’s Modulus (GPa)</td>
<td>13.07</td>
<td>0.76</td>
</tr>
<tr>
<td>Poisson’s Ratio</td>
<td>0.23</td>
<td>0.02</td>
</tr>
<tr>
<td>Internal friction Angle (degree)</td>
<td>41.7</td>
<td>5.6</td>
</tr>
<tr>
<td>Cohesion (MPa)</td>
<td>15.02</td>
<td>5.64</td>
</tr>
</tbody>
</table>

Andesite rock specimens were prepared with 25, 55, 75, and 100 mm diameters. Dimensions of the samples are presented in Fig. 1-A. Special molds are prepared in order to obtain 5 mm uniform liner thickness around the cores. Great care was taken to achieve constant liner thickness. For this purpose, thin sheet persplex layers with circular holes are prepared to place the core and the plastic mold coaxially (Fig. 1-B). By this technique it is ensured that the annulus between the core and the mold is exactly 5 mm (Fig. 1-C).
In order to obtain homogenous test specimens, all the test specimens were prepared in a single process. Mixing is done according to the manufacturer’s recommended ratio by a mixer (Fig. 2-A). After 6 minutes of mixing, fresh mixture is ready for molding. Since there is a limited tack free time after mixing, a silicone cartridge gun was used to inject liner into mold in order to simplify the pouring procedure (Fig. 2-B, 2-C). Molding procedure is done within 15 minutes following the mixing. It should be noted that a thin spatula is used while filling the annulus in order to remove trapped air and to have a homogenous pour distribution.

As can be seen from Figure 3-B, the top and bottom of the lined cores have an offset liner height of 1mm, in order not to load the liner directly during compression tests which might cause a progressive failure of the liner.
After molding the specimens were left to cure for 24 hours and the plastic molds were removed. Since a releasing agent is used for the inner surface of the molds, unmolding the specimens was done with ease. The coated test specimens were cured for 7 days before testing under constant laboratory conditions (about 25°C and 30% humidity). Specimen photographs taken just before testing are presented in Fig. 3-A and B. Liner thickness is measured again just before testing (Fig. 3-C).

In this study, elastic material properties of TSL material are determined. Tensile and deformability tests are performed for 7 day cured samples. To determine tensile properties, ASTM D-638 type I dogbone test specimens are molded and tested. To determine compressive material properties, deformability tests are also performed according to ASTM-D695 test standards. Details of the test procedure and the standards are
presented in Guner, 2014. Elastic material properties of the tested liner are presented in Table 2. Note that since the TSL is a relatively ductile material, no failure state is observed during the deformability tests, so the tests are finalised after observing a linear elastic path in the stress-strain curve.

Table 2  Elastic material properties of tested TSL

<table>
<thead>
<tr>
<th>Property</th>
<th>Value*</th>
<th>Std. Dev.</th>
</tr>
</thead>
<tbody>
<tr>
<td>Tensile Strength (MPa)</td>
<td>2.23</td>
<td>0.13</td>
</tr>
<tr>
<td>Tensile Modulus (MPa)</td>
<td>31.72</td>
<td>6.75</td>
</tr>
<tr>
<td>Modulus of Elasticity (MPa)</td>
<td>45.79</td>
<td>6.57</td>
</tr>
<tr>
<td>Poisson’s Ratio</td>
<td>0.20</td>
<td>0.01</td>
</tr>
</tbody>
</table>

*Tests were performed on 7 day cured specimens

2.3 Test set-up

Laboratory tests are conducted with a displacement controlled MTS-815 machine with a 500kN load cell. Circumferential and axial displacement measurement instruments have a 10 mm capacity with ± 0.001mm sensitivity. During the tests, the system recorded 8 readings per second. Constant displacement rate of 0.3 mm/min was applied and the tests are performed according to ISRM standards (ISRM,1979). Compression test set-up can be seen in Figure 4.

![Test apparatus](image)

1: Axial Extensometers
2: Circumferential Extensometer
3: Load Cell (500kN Capacity)
4: 55 mm diameter sample

Figure 4  Test apparatus

3 Test results

As a result of the laboratory tests, stress-axial strain curves are plotted for each test. The expected strength enhancement could not be observed in most of the tests. Representative stress-strain curves for the coated and the uncoated specimens are presented in Figure 5.
According to Figure 5, as opposed to the intuitive opinion, the TSL has an adverse strength effect on 55, 75mm, and 100 mm diameter samples. For 25 mm samples, the strength improvement is observed due to the confinement effect of the TSL. Representative specimen photos for coated and uncoated test specimens are presented in Figure 6.

In the literature, 6 different failure modes are identified for rocks under uniaxial compression (Basu et al. 2013). In our study, homogenous and isotropic rock samples showed single and y-shaped shear failure modes. For the coated specimens, in addition to these modes, axial splitting failure is also observed. It should be noted that during the tests, rock samples failed before any visual damage is observed on the liner coatings. At the beginning of post failure state, the TSL behaves differently for different sample diameters. The test results are presented in Table 3.
Table 3. Test results

<table>
<thead>
<tr>
<th>Test Condition</th>
<th>Diameter</th>
<th>Test No.</th>
<th>UC S (MPa)</th>
<th>Mean</th>
<th>E (GPa)</th>
<th>Mean</th>
</tr>
</thead>
<tbody>
<tr>
<td>Uncoated</td>
<td>25 mm</td>
<td>1</td>
<td>55.37</td>
<td>51.40 ± 4.55</td>
<td>10.8</td>
<td>10.22 ± 0.70</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>46.43</td>
<td>51.40 ± 4.55</td>
<td>10.42</td>
<td>10.22 ± 0.70</td>
<td></td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>52.39</td>
<td>51.40 ± 4.55</td>
<td>9.44</td>
<td>10.22 ± 0.70</td>
<td></td>
</tr>
<tr>
<td></td>
<td>1</td>
<td>71.54</td>
<td>75.88 ± 4.24</td>
<td>14.08</td>
<td>13.93 ± 0.37</td>
<td></td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>76.08</td>
<td>75.88 ± 4.24</td>
<td>14.08</td>
<td>13.93 ± 0.37</td>
<td></td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>80.02</td>
<td>75.88 ± 4.24</td>
<td>9.44</td>
<td>10.22 ± 0.70</td>
<td></td>
</tr>
<tr>
<td></td>
<td>1</td>
<td>65.89</td>
<td>66.90 ± 5.11</td>
<td>13.51</td>
<td>13.93 ± 0.37</td>
<td></td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>72.44</td>
<td>66.90 ± 5.11</td>
<td>13.51</td>
<td>13.93 ± 0.37</td>
<td></td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>62.38</td>
<td>66.90 ± 5.11</td>
<td>12.71</td>
<td>10.22 ± 0.70</td>
<td></td>
</tr>
<tr>
<td></td>
<td>1</td>
<td>65.04</td>
<td>65.57 ± 2.35</td>
<td>12.1</td>
<td>12.47 ± 0.33</td>
<td></td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>67.5</td>
<td>65.57 ± 2.35</td>
<td>14.27</td>
<td>13.89 ± 0.34</td>
<td></td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>64.18</td>
<td>65.57 ± 2.35</td>
<td>12.1</td>
<td>12.47 ± 0.33</td>
<td></td>
</tr>
<tr>
<td></td>
<td>1</td>
<td>67.25</td>
<td>63.21 ± 3.70</td>
<td>12.24</td>
<td>12.24 ± 0.79</td>
<td></td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>62.43</td>
<td>63.21 ± 3.70</td>
<td>12.01</td>
<td>11.09 ± 1.79</td>
<td></td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>59.96</td>
<td>63.21 ± 3.70</td>
<td>9.02</td>
<td>11.09 ± 1.79</td>
<td></td>
</tr>
<tr>
<td></td>
<td>1</td>
<td>74.50</td>
<td>69.55 ± 5.06</td>
<td>13.78</td>
<td>13.78 ± 0.90</td>
<td></td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>64.38</td>
<td>69.55 ± 5.06</td>
<td>11.98</td>
<td>12.92 ± 0.90</td>
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<tr>
<td></td>
<td>3</td>
<td>69.77</td>
<td>69.55 ± 5.06</td>
<td>12.99</td>
<td>12.92 ± 0.90</td>
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</tr>
<tr>
<td></td>
<td>1</td>
<td>61.24</td>
<td>63.89 ± 2.35</td>
<td>12.1</td>
<td>12.1 ± 0.33</td>
<td></td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>64.69</td>
<td>63.89 ± 2.35</td>
<td>12.72</td>
<td>12.47 ± 0.33</td>
<td></td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>65.75</td>
<td>63.89 ± 2.35</td>
<td>12.58</td>
<td>12.47 ± 0.33</td>
<td></td>
</tr>
<tr>
<td></td>
<td>1</td>
<td>59.07</td>
<td>64.50 ± 5.47</td>
<td>14.72</td>
<td>13.73 ± 0.99</td>
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</tr>
<tr>
<td></td>
<td>2</td>
<td>70.00</td>
<td>64.50 ± 5.47</td>
<td>14.72</td>
<td>13.73 ± 0.99</td>
<td></td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>64.45</td>
<td>64.50 ± 5.47</td>
<td>12.74</td>
<td>12.74 ± 0.99</td>
<td></td>
</tr>
</tbody>
</table>

As a result of this study, for 25 mm diameter coated samples, 23% strength enhancement is observed. Moreover, prior to failure, coated samples sustained greater displacements compared to the uncoated samples. However, for 55 and 75 mm diameter samples the coated samples had relatively less load bearing capacity compared to the uncoated ones. The 100 mm diameter sample had almost no strength enhancement or weakening for pre-failure state.

In addition, elasticity moduli values are calculated for the deformability tests. As presented in Table 3, a slight softening effect for all tests except for the 25 mm diameter sample was observed (7.2% to 1.1%). As diameter of the sample increases, softening effect decreases. Excluding the 25 mm diameter sample, uniaxial strength loss and strength softening effect has almost the same change.
4 Discussion & conclusions

In this study, the effect of specimen diameter on a fixed thickness coated compression test is investigated. Coated compression testing of rock core is a commonly used test methodology for estimating pillar reinforcement effects of TSL’s. Although the size of the coated compression test specimens may not represent the large scale pillar behaviour, the support mechanism of the TSL-coated materials is assumed to be same. For this purpose, different diameter cylindrical samples (25, 55, 75, 100 mm) are prepared and tested in uncoated and 5 mm coated conditions under exactly the same laboratory conditions to simulate different pillar sizes. In addition to this, deformability and tensile tests are performed for the liner. As pillar sizes are very high compared to test samples, intuitively the confinement effect of TSL’s would be very slight or none. One of the major purposes of this study is to show this relationship.

The previous researchers have reported that for weaker rocks (UCS <50 MPa), TSL’s have higher reinforcement effect compared to harder rocks (Qiao et al., 2014) for NX core size (54 mm). In our study, tested andesite samples are in a harder rock strength range. However micro and macro scale cracks, grain size and porosity of the tested specimens are as important as the hardness of the rocks. Although it is expected to get some strength enhancement even for larger scale samples, we observe a worsening effect TSL’s have on UCS tests of core samples.

We observed two different effects of TSLs; strength enhancement due to confinement and strength reduction which we have attributed to weakening of the rock samples by moisture absorption. For 25 mm samples, we can clearly see the confinement effect of TSL’s where the rock core gets a 23% strength gain. On the other hand, for the other samples, the worsening effect is the dominant due to the porous nature of the andesite samples. The moisture absorption of the sample may lead to 8.3% to 1.6% strength reductions. As sample diameter increases, since the moistened area of the specimen becomes small compared to the cross-sectional area of the specimen, the strength reduction effect decreases.

Although TSL coatings do not have any apparent effect in pre failure region of the samples, it is estimated that the major enhancement might be observed in post failure region. This needs to be further analysed by post failure behaviour of cores with different porosity.

Acknowledgement

We would like to thank to Tahsin Işıkşal, for his help and guidance during specimen preparation work.

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Scattering of SH-waves by a shallow circular lined tunnel with an imperfect interface

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Abstract

The analytic solutions for the dynamic response of a shallow circular lined tunnel with an imperfectly bonded interface subjected to plane SH-waves are presented in the paper. Complex variable method was used and the imperfect interface was modelled with a linear spring model. The case that the rock is harder than the liner was investigated. The effects of the contact stiffness of the interface, the incident angle, the frequency of the incident wave and the depth of the tunnel were investigated. The results indicate when the frequency of incident waves is low, the variation of contact stiffness of the imperfect interface has a slight effect on the distribution of dynamic stress concentration factor (DSCF) in the rock mass but there is a significant effect on the distribution of DSCF in the liner. When the frequency of incident waves is high, the distribution of DSCF is complicated in the rock mass and in the liner. The variation of the depth of tunnel leads to the cyclical variation of DSCF. The incident angle significantly affects the distribution and value of DSCF. The phenomenon of resonance scattering can be observed when the bond of the interface is extremely weak.

1 Introduction

Underground structures play an important role in the infrastructure of modern society, since they are involved in a number of applications ranging from storage to transportation systems. It is generally assumed that underground structures suffer appreciably less damage under seismic conditions than structures on the surface and, in particular, that such damage diminishes with increasing overburden depth (Hashash et al., 2001). However, some tunnels were seen to suffer damage beyond the limits of possible refurbishment during earthquakes (Uenishi and Sakurai 2000; Li 2012). In order to provide a safe condition for the underground structure, it is necessary to design the support system of underground facilities to withstand static overburden loads as well as to accommodate the additional deformations imposed by the earthquake induced motions (Hasheminejad and Miri 2008). Consequently, development of a suitable analysis as part of the design methodology for assessment of the dynamic interaction effects between the lining and its surrounding media when subjected to seismic waves is crucial for earthquake and civil engineers (Hashash et al., 2005).

Extensive efforts have been dedicated to studying the dynamic response of lined or unlined tunnels. Using the wave function expansion method, Pao and Mow (1973) initiated the study of dynamic stress concentration of single cavity in the whole space for incident elastic plane P waves. Lee (1977) used complex variable solution for incident SH wave to cylindrical cavity. Hwang and Lysmer (1981) used a special FEM in frequency domain for dynamic analysis of buried structures to plane travelling wave. Datta and Shah (1982) undertook a study on wave scattering around single or multiple cavities. The series expansion method was used by Zeng and Cakmak (1985) to investigate the scattering of plane SH waves by multiple cavities in both an infinite and a half space. Kattis et al. (2003) studied 2D dynamic response of unlined and lined tunnels in poroelastic soil to harmonic body waves. Dynamic response of twin lined
tunnels buried in an infinite medium and subjected to seismic loadings was investigated by Moore and Guan (1996) using the successive reflection method.

The effect of surface topography on seismic wave propagation is an important topic for earthquake engineering. Liang et al. (2003, 2004) used Fourier–Bessel expansions to derive a series solution of the displacement response of the ground surface in the presence of underground twin tunnels subjected to excitation of incident plane SV and P waves. In their works, the half surface was replaced by a convex circular surface of large radius. Qi et al. (2003) investigated the dynamic response of shallow lined structure by incident SH-wave using multi polar coordinates system method. Liu et al. (2013) presented an analytical solution for scattering of plane harmonic P, SV or Rayleigh waves by a shallow lined circular tunnel in an elastic half space based on the plane complex variable theory and the image technique.

Generally, the interface between the tunnel and the liner is treated as perfectly bonded in previous studies, which means that the traction and displacement on the interface are continuous. In practice, interface bonding is often imperfect because of the presence of microcracks or interstitial media in the interface. It is therefore of technological interest to study to what extent the response of the system is affected by the imperfect bonding between the liner and the surrounding rock. Yi et al (2014a) investigated the effect of imperfect interface on the dynamic response of lined tunnel under incident cylindrical P-waves by using a spring model to model the contact between the tunnel and the liner. Yi et al. (2014b) also investigated the effect of imperfect interface on the dynamic response of lined tunnel under incident SH-waves in a whole space.

The primary goal of the paper is to study the scattering of SH-waves by a shallow circular lined tunnel with an imperfect interface. Here, a shallow lined tunnel means the lined tunnel is in a half space, i.e., the effect of the ground surface should be taken into account. In this paper, the effects of the contact stiffness of the interface, the incident angle, the frequency of the incident wave and the depth of the tunnel on the dynamic response of the lined tunnel are investigated.

2 Governing equations

2.1 SH-waves in solid

In xy-plane, the motion \( W(x,y,t) \) excited by SH-wave is normal to xy-plane and independent of \( z \). It is governed by the two-dimensional wave equation.

\[
\frac{\partial^2 W}{\partial x^2} + \frac{\partial^2 W}{\partial y^2} = \frac{1}{C_s^2} \frac{\partial^2 W}{\partial t^2}
\]

where, \( C_s = \sqrt{\frac{\mu}{\rho}} \) is the shear wave velocity, \( \rho \) and \( \mu \) are mass density and shear modulus, respectively.

For the case of a harmonic SH-wave, assuming \( W = w(x, y)e^{-i\omega t} \), eq. (1) can be rewritten as:

\[
\frac{\partial^2 w}{\partial x^2} + \frac{\partial^2 w}{\partial y^2} + \beta^2 w = 0
\]

where, \( w \) is the displacement, \( \beta = \omega/c_s \) is the wave number of the incident wave, \( \omega = 2\pi f \) is the circular frequency and \( f \) is the frequency of the incident wave.

After omitting the time factor of \( e^{-i\omega t} \), the relationship between stress and strain is as follows

\[
\tau_{xx}(x, y) = \mu \frac{\partial w}{\partial x}, \quad \tau_{yy}(x, y) = \mu \frac{\partial w}{\partial y}
\]
By introducing a complex variable \( z = x + yi \), \( \bar{z} = x - yi \), \( i = \sqrt{-1} \), in the complex plane \( z, \bar{z} \), eqs. (2) and (3) can be written as

\[
\frac{\partial^2 w}{\partial z \partial \bar{z}} + \frac{1}{4} \beta^2 w = 0 \tag{4}
\]

\[
\tau_{xz} = \mu \left( \frac{\partial w}{\partial z} + \frac{i \mu}{\partial \bar{z}} \right), \quad \tau_{yz} = i \mu \left( \frac{\partial w}{\partial z} - \frac{i \mu}{\partial \bar{z}} \right) \tag{5}
\]

In the polar coordinates, eq. (5) can be written as

\[
\tau_{r\theta} = \mu \left( \frac{\partial w}{\partial r} e^{i\theta} + \frac{i \mu}{\partial \theta} e^{-i\theta} \right), \quad \tau_{\theta\theta} = i \mu \left( \frac{\partial w}{\partial r} e^{i\theta} - \frac{i \mu}{\partial \theta} e^{-i\theta} \right) \tag{6}
\]

### 2.2 Waves in rock mass

We consider an infinitely long lined circular tunnel of outer radius \( b \) and inner radius \( a \) in the rock mass. A plane harmonic SH-wave propagates in the rock mass with incident angle of \( \alpha_0 \) and meets the lined tunnel, see Figure 1. In complex plane \( z, \bar{z} \), the incident SH-wave can be expressed as (Lin and Liu 2002)

\[
W_{1}^{(i)} = W_0 e^{i \beta_{i} \left( (z - i h) e^{-i\alpha_0} + (\bar{z} + i h) e^{i\alpha_0} \right)} \tag{7}
\]

where \( W_{1}^{(i)} \) is the incident SH-wave, \( W_0 \) is the amplitude of the incident wave, \( \beta_{i} \) is the wave number of the SH-wave in the rock mass. \( h \) is the distance \( OO_1 \) in Figure 1.

The corresponding stresses are expressed as

\[
\tau_{r1}^{(i)} = i \tau_0 \cos(\theta - \alpha_0) e^{i \beta_{i} \left( (z - i h) e^{-i\alpha_0} + (\bar{z} + i h) e^{i\alpha_0} \right)} \tag{8}
\]

\[
\tau_{\theta1}^{(i)} = -i \tau_0 \sin(\theta - \alpha_0) e^{i \beta_{i} \left( (z - i h) e^{-i\alpha_0} + (\bar{z} + i h) e^{i\alpha_0} \right)} \tag{9}
\]

where \( \tau_0 = \beta_{i} \mu_1 W_0 \) is the maximum stress induced by SH-wave in the rock mass. \( \mu_1 \) is the shear modulus of the rock mass.

There is a reflected wave when the incident wave meets the ground surface. It can be written in the complex plane \( z, \bar{z} \) as
\[ W_1^{(r)} = W_0 e^{i\frac{2i}{\beta_1} (z-i\delta) e^{i\omega t} + (z+i\delta) e^{-i\omega t}} \]  

The corresponding stresses are

\[ \tau_{rc1}^{(r)} = i\tau_0 \cos(\theta + \alpha_0) e^{i\frac{2i}{\beta_1} (z-i\delta) e^{i\omega t} + (z+i\delta) e^{-i\omega t}} \]  

\[ \tau_{\theta c1}^{(r)} = -i\tau_0 \sin(\theta + \alpha_0) e^{i\frac{2i}{\beta_1} (z-i\delta) e^{i\omega t} + (z+i\delta) e^{-i\omega t}} \]

There is a reflected wave from the interface when the incident wave meets the liner, it can be written in the complex plane \((z, \bar{z})\)

\[ w_1^{(s)} = \sum_{m=-\infty}^{\infty} A_m \left( H_{m}^{(1)}(\beta_1 | z) \left[ \frac{\bar{z}}{|z|} \right]^m + H_{m}^{(1)}(\beta_1 | z-2hi) \left[ \frac{\bar{z}-2hi}{|z-2hi|} \right]^m \right) \]  

where \(A_m\) is an uncertain constants and \(H_{m}^{(1)}\) is the Hankel function of the first kind for integer order \(m\), which represents an outward-propagating cylindrical wave.

The corresponding stresses are:

\[ \tau_{rc1}^{(s)} = \frac{\beta_1 \mu_1}{2} \sum_{m=-\infty}^{\infty} A_m \left( H_{m+1}^{(1)}(\beta_1 | z) \left[ \frac{\bar{z}}{|z|} \right]^{m+1} e^{i\omega t} - H_{m+1}^{(1)}(\beta_1 | z) \left[ \frac{\bar{z}}{|z|} \right]^{m+1} e^{-i\omega t} \right. 
\]

\[ + \left\{ -H_{m+1}^{(1)}(\beta_1 | z-2hi) \left[ \frac{\bar{z}-2hi}{|z-2hi|} \right]^{(m+1)} e^{i\omega t} + H_{m+1}^{(1)}(\beta_1 | z-2hi) \left[ \frac{\bar{z}-2hi}{|z-2hi|} \right]^{(m+1)} e^{-i\omega t} \right\} \]  

\[ \tau_{\theta c1}^{(s)} = \frac{\beta_1 \mu_1}{2} \sum_{m=-\infty}^{\infty} A_m \left( H_{m+1}^{(1)}(\beta_1 | z) \left[ \frac{\bar{z}}{|z|} \right]^{m+1} e^{i\omega t} + H_{m+1}^{(1)}(\beta_1 | z) \left[ \frac{\bar{z}}{|z|} \right]^{m+1} e^{-i\omega t} \right. 
\]

\[ + \left\{ -H_{m+1}^{(1)}(\beta_1 | z-2hi) \left[ \frac{\bar{z}-2hi}{|z-2hi|} \right]^{(m+1)} e^{i\omega t} - H_{m+1}^{(1)}(\beta_1 | z-2hi) \left[ \frac{\bar{z}-2hi}{|z-2hi|} \right]^{(m+1)} e^{-i\omega t} \right\} \]  

The total displacement field in the rock mass is

\[ W_1^{(T)} = W_1^{(i)} + W_1^{(r)} + W_1^{(s)} \]  

The total stress field in the rock mass is

\[ \tau_{rc1}^{(T)} = \tau_{rc1}^{(i)} + \tau_{rc1}^{(r)} + \tau_{rc1}^{(s)} \]  

\[ \tau_{\theta c1}^{(T)} = \tau_{\theta c1}^{(i)} + \tau_{\theta c1}^{(r)} + \tau_{\theta c1}^{(s)} \]

### 2.3 Waves in liner

There is an SH-wave \((w_2^{(i)})\) that propagates to the inside of the liner from the outside boundary of liner. It can be expressed as:

\[ w_2^{(i)} = \sum_{m=-\infty}^{\infty} B_m \left\{ H_m^{(2)}(\beta_2 | z) \left[ \frac{z}{|z|} \right]^m \right\} \]  

where \(B_m\) is an uncertain constant and \(H_m^{(2)}\) is the Hankel function of the second kind for integer order \(m\), which represents an inward-propagating cylindrical wave. \(\beta_2\) is the wave number of the SH-wave in the liner.
The corresponding stresses are:

$$
\tau_{rz}^{(t)} = \frac{\beta_2 \mu_2}{2} \sum_{m=-\infty}^{\infty} B_m \{ H_m^{(2)}(\beta_2 | z |) \left( \frac{z}{|z|} \right)^{m-1} e^{i\theta} - H_{m+1}^{(2)}(k_2 | z |) \left( \frac{z}{|z|} \right)^{m+1} e^{-i\theta} \} 
$$

(20)

$$
\tau_{\theta z}^{(t)} = i \beta_2 \mu_2 \sum_{m=-\infty}^{\infty} B_m \{ H_m^{(1)}(\beta_2 | z |) \left( \frac{z}{|z|} \right)^{m-1} e^{i\theta} + H_{m+1}^{(1)}(\beta_2 | z |) \left( \frac{z}{|z|} \right)^{m+1} e^{-i\theta} \} 
$$

(21)

where \( \mu_2 \) is the shear modulus of the liner.

There is a reflected SH-wave in the liner which propagates outwards from the inside boundary of the liner. It can be expressed as

$$
W_2^{(r)} = \sum_{m=-\infty}^{\infty} C_m \left\{ H_m^{(1)}(\beta_2 | z |) \left( \frac{z}{|z|} \right)^m \right\} 
$$

(22)

The corresponding stresses are:

$$
\tau_{rz}^{(r)} = \frac{\beta_2 \mu_2}{2} \sum_{m=-\infty}^{\infty} C_m \left\{ H_m^{(1)}(\beta_2 | z |) \left( \frac{z}{|z|} \right)^{m-1} e^{i\theta} - H_{m+1}^{(1)}(\beta_2 | z |) \left( \frac{z}{|z|} \right)^{m+1} e^{-i\theta} \right\} 
$$

(23)

$$
\tau_{\theta z}^{(r)} = i \beta_2 \mu_2 \sum_{m=-\infty}^{\infty} C_m \left\{ H_m^{(1)}(\beta_2 | z |) \left( \frac{z}{|z|} \right)^{m-1} e^{i\theta} + H_{m+1}^{(1)}(\beta_2 | z |) \left( \frac{z}{|z|} \right)^{m+1} e^{-i\theta} \right\} 
$$

(24)

The total displacement field and the stress field in the liner are

$$
W_2^{(T)} = W_2^{(t)} + W_2^{(r)}
$$

(25)

$$
\tau^{(T)}_{rz} = \tau^{(t)}_{rz} + \tau^{(r)}_{rz}
$$

(26)

$$
\tau^{(T)}_{\theta z} = \tau^{(t)}_{\theta z} + \tau^{(r)}_{\theta z}
$$

(27)

### 3 Boundary conditions

The support system such as liner usually is designed for underground facilities, where the liners generally are neither perfectly bonded to nor perfectly lubricated from the surrounding rock, i.e. the interface is usually an imperfect bonding. Several imperfect interface models were proposed (Jones and Whittier, 1967; Murty, 1975; Newmark et al., 1951; Paskaramoorthy et al., 1988; Rokhlin and Wang, 1991). The linear spring model (Newmark et al., 1951) adopted for this work is one of the popular models for modeling the imperfect interface (Achenbach and Zhu, 1989; Sudak et al., 1999; Shen et al., 2001; Valier-Brasier et al., 2012). The model assumes that tractions are continuous but displacements may be discontinuous across the interface. More precisely, jumps in the displacement components are assumed to be proportional to their respective interface traction components. These interface parameters are assumed to be uniform along the entire length of the interface and the interface model is said to represent a homogeneously imperfect interface. Using this concept, the boundary conditions to be applied at the interface of the liner and the rock mass can be described as:

$$
u_{zt} - u_{zt} = \frac{\sigma_{rzt}}{k_s}
$$

(28a)

$$
\sigma_{rzt} = \sigma_{rzt}
$$

(28b)
\[
\sigma_{rz} = 0 \quad \text{(29)}
\]

\[
\sum_{m=-\infty}^{\infty} (A_m \zeta_m^{(11)} + B_m \zeta_m^{(12)} + C_m \zeta_m^{(13)}) = \zeta^{(1)} \quad \text{(30)}
\]

Where:

\[
\zeta_m^{(11)} = H_m^{(1)}(\beta_1 | z|) \left( \frac{z}{|z|} \right)^m + H_m^{(1)}(\beta_2 | z|) \left( \frac{z - 2hi}{|z - 2hi|} \right)^m
\]

\[
- \frac{\beta_1 \mu_1}{2k_s} \{ H_{m-1}^{(1)}(\beta_1 | z|) \left( \frac{z}{|z|} \right)^{m-1} e^{i\theta} - H_{m+1}^{(1)}(\beta_1 | z|) \left( \frac{z}{|z|} \right)^{m+1} e^{-i\theta} + H_{m+1}^{(1)}(\beta_1 | z - 2hi|) \left( \frac{z - 2hi}{|z - 2hi|} \right)^{m+1} e^{-i\theta} \}
\]

where \( k_s \) is tangential spring stiffness. The subscripts of 1 and 2 denote the components in the rock and the liner, respectively.

At the inner boundary of the liner, the boundary condition can be expressed as:

Substituting eqs. (17), (18) and (25) into eq.(28a), we can get

\[
\zeta_m^{(12)} = -H_m^{(2)}(\beta_2 | z|) \left( \frac{z}{|z|} \right)^m
\]

\[
\zeta_m^{(13)} = -H_m^{(1)}(\beta_2 | z|) \left( \frac{z}{|z|} \right)^m
\]

\[
\zeta^{(1)} = \frac{i \beta_s \mu_2 W_0}{k_s} \{ \cos(\theta - \alpha_0) e^{i \beta_s (z - ih)e^{i\theta} + (\bar{z} + ih)e^{i\theta}} + \cos(\theta + \alpha_0) e^{i \beta_s (z - ih)e^{i\theta} + (\bar{z} + ih)e^{i\theta}} \}
\]

- \( W_0 e^{i \beta_s (z - ih)e^{i\theta} + (\bar{z} + ih)e^{i\theta}} - W_0 e^{i \beta_s (z - ih)e^{i\theta} + (\bar{z} + ih)e^{i\theta}} \)

Substituting eqs. (17) and (26) into the second term of eq. (28b), we can get

\[
\sum_{m=-\infty}^{\infty} (A_m \zeta_m^{(21)} + B_m \zeta_m^{(22)} + C_m \zeta_m^{(23)}) = \zeta^{(2)} \quad \text{(31)}
\]

where

\[
\zeta_m^{(21)} = \frac{\beta_2 \mu_2}{2} \{ H_{m-1}^{(1)}(\beta_2 | z|) \left( \frac{z}{|z|} \right)^{m-1} e^{i\theta} - H_{m+1}^{(1)}(\beta_2 | z|) \left( \frac{z}{|z|} \right)^{m+1} e^{-i\theta} + H_{m+1}^{(1)}(\beta_2 | z - 2hi|) \left( \frac{z - 2hi}{|z - 2hi|} \right)^{m+1} e^{-i\theta} \}
\]

\[
\zeta_m^{(22)} = -\frac{\beta_2 \mu_2}{2} \{ H_{m}^{(2)}(\beta_2 | z|) \left( \frac{z}{|z|} \right)^{m} e^{i\theta} - H_{m+1}^{(2)}(\beta_2 | z|) \left( \frac{z}{|z|} \right)^{m+1} e^{-i\theta} \}
\]

\[
\zeta_m^{(23)} = -\frac{\beta_2 \mu_2}{2} \{ H_{m}^{(1)}(\beta_2 | z|) \left( \frac{z}{|z|} \right)^{m} e^{i\theta} - H_{m+1}^{(1)}(\beta_2 | z|) \left( \frac{z}{|z|} \right)^{m+1} e^{-i\theta} \}
\]
\[ \zeta^{(2)} = -i \beta \mu W_0 \left\{ \cos(\theta - \alpha_0) e^{i \beta (z - ih) \cos(\theta - \alpha_0)} + \cos(\theta + \alpha_0) e^{i \beta (z + ih) \cos(\theta + \alpha_0)} \right\} \]

According to Eq. (29), we can get

\[ \sum_{m=-\infty}^{\infty} \left( B_m \zeta^{(32)}_m + C_m \zeta^{(33)}_m \right) = 0 \]  

(32)

where

\[ \zeta^{(32)}_m = \frac{\beta_2 \mu_2}{2} \left\{ H^{(2)}_{m-1} (\beta_2 | z|) \left[ \frac{Z}{|z|} \right]^{m+1} e^{i \theta} - H^{(2)}_{m+1} (\beta_2 | z|) \left[ \frac{Z}{|z|} \right]^{m+1} e^{-i \theta} \right\} \]

\[ \zeta^{(33)}_m = \frac{\beta_3 \mu_3}{2} \left\{ H^{(1)}_{m-1} (\beta_3 | z|) \left[ \frac{Z}{|z|} \right]^{m+1} e^{i \theta} - H^{(1)}_{m+1} (\beta_3 | z|) \left[ \frac{Z}{|z|} \right]^{m+1} e^{-i \theta} \right\} \]

Then, both sides of eqs. (30), (31) and (32) are multiplied by \( e^{-in \theta} \) and integrated from \(-\pi\) to \(\pi\). Thus, the problem is reduced to a series of equations as follows:

\[ \sum_{m=-\infty}^{\infty} \begin{pmatrix} \zeta^{(11)}_{mn} \\
\zeta^{(12)}_{mn} \\
\zeta^{(13)}_{mn} \\
0 \\
\zeta^{(21)}_{mn} \\
\zeta^{(22)}_{mn} \\
\zeta^{(23)}_{mn} \\
0 \\
\zeta^{(31)}_{mn} \\
\zeta^{(32)}_{mn} \\
\zeta^{(33)}_{mn} \\
0 \\
A_m \\
B_m \\
C_m \end{pmatrix} \begin{pmatrix} A_n \\
B_n \\
C_n \end{pmatrix} = \begin{pmatrix} \zeta^{(1)}_n \\
\zeta^{(2)}_n \\
\zeta^{(3)}_n \end{pmatrix} \]

(33)

where

\[ \zeta^{(11)}_{mn} = \frac{1}{2 \pi} \int_{-\pi}^{\pi} \zeta^{(11)}_m e^{-i n \theta} d\theta \]

\[ \zeta^{(12)}_{mn} = \frac{1}{2 \pi} \int_{-\pi}^{\pi} \zeta^{(12)}_m e^{-i n \theta} d\theta \]

\[ \zeta^{(13)}_{mn} = \frac{1}{2 \pi} \int_{-\pi}^{\pi} \zeta^{(13)}_m e^{-i n \theta} d\theta \]

\[ \zeta^{(21)}_{mn} = \frac{1}{2 \pi} \int_{-\pi}^{\pi} \zeta^{(21)}_m e^{-i n \theta} d\theta \]

\[ \zeta^{(22)}_{mn} = \frac{1}{2 \pi} \int_{-\pi}^{\pi} \zeta^{(22)}_m e^{-i n \theta} d\theta \]

\[ \zeta^{(23)}_{mn} = \frac{1}{2 \pi} \int_{-\pi}^{\pi} \zeta^{(23)}_m e^{-i n \theta} d\theta \]

\[ \zeta^{(31)}_{mn} = \frac{1}{2 \pi} \int_{-\pi}^{\pi} \zeta^{(31)}_m e^{-i n \theta} d\theta \]

\[ \zeta^{(32)}_{mn} = \frac{1}{2 \pi} \int_{-\pi}^{\pi} \zeta^{(32)}_m e^{-i n \theta} d\theta \]

\[ \zeta^{(33)}_{mn} = \frac{1}{2 \pi} \int_{-\pi}^{\pi} \zeta^{(33)}_m e^{-i n \theta} d\theta \]

Therefore, the unknown coefficients of \( A_m, B_m \) and \( C_m \) can be determined by solving eq. (33).

4 Numerical results and discussions

Dynamic effects on lined tunnels are in the form of stress concentrations or deformations that they experience when suffering dynamic loads. It is important to solve the DSCF in the tunnel and the liner under the incident SH-wave. For this problem the main task is to study the DSCF in the rock mass and the edge of the liner. The DSCF \( \tau^*_{\theta z} \) can be expressed as

\[ \tau^*_{\theta z} = \left| \tau_{\theta z} / \tau_0 \right| \]

(34)

To get a general solution, some dimensionless parameters are defined. We define that the ratio of the wave numbers of the liner and the rock \( \beta_1 / \beta = 1.5 \) and the ratio of the shear moduli of the rock and the liner is \( \mu_1 / \mu_2 = 2.9 \). This case can be regarded as a concrete liner in rock. We define three spring stiffness values \( K_1 = 0.1 \beta_1 \mu_1, 1.0 \beta_1 \mu_1, 10.0 \beta_1 \mu_1 \) to study the effect of imperfect interface. The ratio of outer radius to inner radius of the liner \( b/a \) is 1.2.
4.1 Effect of imperfect interface

To investigate the effect of imperfect interface, the incident angle is set as $90^\circ$ and the ratio of the depth of the tunnel and the inner radius of the liner $h/a=1.5$. The distributions of DSCF are shown in Figure 2. When the frequency of the incident wave is low, the maximum of DSCF is at around $\theta=210^\circ$ and $330^\circ$ in the rock and the liner, see Figure 2 (a) and Figure 2 (b). Increasing spring stiffness leads to slightly decreasing DSCF in the rock and increasing DSCF in the liner. When the frequency of the incident wave is high, the distribution of DSCF is complicated in both rock and liner, see Figure 2 (c) and Figure 2 (d). Three different spring stiffness values lead to similar DSCF in the rock and much different DSCF in the liner. The comparison of Figure 2 (a) and Figure 2 (c) shows that the low frequency incident wave leads to smaller DSCF in the rock than the high frequency incident wave, which is different from the situation that the tunnel is in a whole space (Yi et al. 2014b). So the ground surface is another important factor for the distribution of DSCF.

4.2 Effect of depth of tunnel

To investigate the effect of depth of the tunnel on the dynamic response of lined tunnel, only the ratio of the depth of the tunnel and the inner radius of the liner $h/a$ is changed to 30 in this case compared to the case in section 4.1. The distributions of DSCF are shown in Figure 3. When the frequency of the incident wave is low, the maximum of DSCF is at around $\theta=30^\circ$ and $150^\circ$ in the rock and the liner, see Figure 3 (a) and Figure 3 (b), which is quite different from the low frequency case in section 4.1. When $K_s=0.1\beta_i\mu_i$, the
contact between the rock mass and the liner is weak, which induces the DSCF in the liner is quite small compared to $K = 1.0\beta\mu_i$ and $K = 10.0\beta\mu_i$, see Figure 3 (b).

When the frequency of the incident wave is high, there are four peak values of DSCF in the rock and the liner, see Figure 3 (c) and Figure 3 (d). The variation of spring stiffness has a slight effect on the distribution of DSCF in the rock, see Figure 3 (c). But in the liner, $K = 1.0\beta\mu_i$ induces the largest DSCF while $K = 0.1\beta\mu_i$ induces the smallest DSCF because of the weak contact.

Figure 4 shows the variation of DSCF at $\theta=0^\circ$ with the depth change when $K = 10.0\beta\mu_i$ and the incident angle is $90^\circ$. The results indicate that the DSCF at $\theta=0^\circ$ changes cyclically with the variation of the depth of the tunnel. The DSCF varies sharply for high frequency incident waves while it varies smoothly for low frequency incident waves.
Scattering of SH-waves by a shallow circular lined tunnel with an imperfect interface

C.P. Yi, D. Johansson, U. Nyberg

Ground Support 2016, Luleå, Sweden

4.3 Effect of incident angle

To investigate the effect of depth of the tunnel on the dynamic response of lined tunnel, only the incident angle $\alpha_0$ is changed to 45° in this case compared to the case in section 4.1. The distributions of DSCF are shown in Figure 5. When the frequency of the incident wave is low, the maximum DSCF is at 90° in the rock and the liner, see Figure 5 (a) and Figure 5 (b). Increasing spring stiffness leads to slightly decreasing DSCF in the rock and increasing DSCF in the liner. By comparing Figure 5 (a) to Figure 2 (a) and comparing Figure 5 (b) to Figure 2 (b), it can be observed that the maximum DSCF in both rock and liner for incident angle of 45° is much greater than that for the incident angle of 90° when $h/a$ is 1.5 and the frequency of the incident wave is low, though the maximum DSCF is located at different positions due to different incident angle.

When the frequency of the incident wave is high, the distribution of DSCF is complicated both in the rock and the liner, see Figure 5 (c) and Figure 5 (d). The variation of spring stiffness has a slight effect on the distribution of DSCF in the rock; see Figure 5 (c). But in the liner, $K_s = 1.0\beta_1\mu_1$ induces the largest DSCF in the liner while $K_s = 0.1\beta_1\mu_1$ induces the smallest DSCF in the liner because of the weak contact.

Figure 6 shows the variation of DSCF at $\theta = 90^\circ$ with the frequency change and the contact stiffness change when the incident angle is 45°. $K_s = 1.0\beta_1\mu_1$ and $K_s = 10.0\beta_1\mu_1$ give similar DSCF variation curves with the change of frequency for both rock and liner. When the bond is extremely imperfect ($K_s = 0.1\beta_1\mu_1$), the results show that there are several peak values of DSCF in the rock and the liner, which is due to the resonance scattering as observed by Rajabi et al. (2009). This phenomenon is very unique for the case of the extremely imperfect interface (Wang and Sudak 2007).
Figure 5  DSCF in the rock and the liner with $\alpha_0 = 45^\circ$ and $h/a=1.5$ (a) DSCF in the rock at $r=b$ with low frequency incident waves; (b) DSCF in the liner at $r=a$ with low frequency incident waves; (c) DSCF in the rock at $r=b$ with high frequency incident waves; (d) DSCF in the liner at $r=a$ with high frequency incident waves.

Figure 6  DSCF versus the frequency of incident wave with $\alpha_0 = 45^\circ$ (a) Variation of DSCF in the rock at $r=b$ and $\theta=90^\circ$ (b) Variation of DSCF in the liner at $r=a$ and $\theta=90^\circ$. 
5 Conclusion

1. The dynamic response of a shallow lined tunnel with an imperfect interface under incident SH-waves was studied by using the complex variable method and the spring model for the imperfect interface. Some observations are as follows,

2. When the frequency of incident waves is low, the variation of contact stiffness has a slight effect on the distribution of dynamic stress concentration factor (DSCF) in the rock mass but there is a significant effect on the distribution of DSCF in the liner.

3. When the frequency of incident waves is high, the distribution of DSCF is complicated in the rock mass and the liner.

4. The variation of the depth of tunnel leads to the cyclical variation of DSCF. DSCF changes sharply when the frequency of incident waves is high while it changes smoothly when the frequency of incident waves is low.

5. The maximum DSCF in both rock and liner for incident angle of 45°is much greater than that for the incident angle of 90° when $h/a$ is 1.5 and the frequency of the incident wave is low.

6. The phenomenon of resonance scattering can be observed when the bond of an interface is extremely weak.

References


Liu, Q, Zhao, M & Wang, L 2013, ´Scattering of plane P, SV or Rayleigh waves by a shallow lined tunnel in an elastic half space’. Soil Dynamics and Earthquake Engineering, 49: 52–63.


Selection of ground support for mining drives based on the Q-System

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Abstract
As part of the Ground Support Systems Optimisation (GSSO) project undertaken by the Australian Centre for Geomechanics and collaborators, a comprehensive review of the current ground support design approaches implemented at mine sites throughout Australia and Canada was conducted. The review has shown that most mines rely primarily on the Grimstad and Barton (1993) chart to select their initial ground support patterns and standard ground support practices.

There are a number of issues and limitations relating to the application to mining problems of this empirical technique, originally developed for civil engineering. These limitations are to a large degree attributable to the absence of mining case studies in the database and changes in ground support practices since the development of the original guidelines. As part of the GSSO research project a significant database of ground support practices used at mine sites has recently been developed. New empirical guidelines, mainly based on mining data collected from Australia and Canada, are proposed to be used at the feasibility stage, as a first pass design and for subsequent optimisation.

1 Introduction

The selection and design of ground support for underground excavations in rock can be achieved through analytical, empirical and numerical techniques. The advantages and limitations of the various methods have been widely represented in literature. In practice it is realised that a design engineer may use all available tools at his or her disposal. The major challenges are the availability of quality rock mass data and understanding the limitations inherent in the assumptions and simplifications necessary to construct a design model whether analytical, empirical or numerical.

The reality is that empirical techniques are most prevalent particularly at the early stages of a project. Empirical techniques can be a set of rules based on experience, or more often a set of recommendations based on the rock mass quality as determined by use of a rock mass classification system. There are several rock mass classification systems available and their advantages and limitations have been widely debated elsewhere, e.g. Milne et al. (1998), Palmstrom & Broch (2006), Potvin et al. (2012). Further detailed discussion on the merits of these systems is beyond the scope of this paper. These rock mass classification systems have been associated with guidelines linking rock mass quality to the selection and design of rock reinforcement and support.

Although the rock mass classification systems, irrespective of their origins, are widely used for a range of engineering applications, it makes sense that every engineering project has its own specificity and design tolerance. Consequently, any ground support recommendations have to be reviewed in the specific context of a project.

This paper focuses on the use of one rock mass classification system, the Q-system, originally proposed by Barton et al. (1974), and provides a series of ground support guidelines for mine drives. These new recommendations are based on an extensive review of ground control practices in North American and Australian mines.
2 Rock mass classification systems for ground support

Rock mass classification systems are used widely in rock engineering for a variety of applications. Over the years there has been a tendency to use the results of rock mass classification systems for a variety of purposes well beyond the limits of their constitutive databases. At the same time there have been numerous publications that identify the limitations of classification system choices and pertinence of the constitutive parameters, the assigned weight to the different parameters and even the concept of an index or a unique value that captures the rock mass behaviour.

Rock mass classification systems are the link between rock mass quality and the choice of support. An important assumption of these systems is that the classification adequately captures the rock mass conditions. Terzaghi’s (1946) work is of interest in that it was the first classification system proposed and it recognised the importance of geological structure “from an engineering point of view, knowledge of the type and intensity of the rock defects may be more important than that of the types of rock which will be encountered.” Terzaghi’s system provided guidelines for estimating the loads to be supported by steel arches in tunnels. In the 1970’s, a number of new and more elaborate classification systems were developed (Table 1) with similar objectives: “…enables a designer to relate the experiences on rock conditions and support requirements gained on other sites to the conditions anticipated on his own site” (Hoek & Brown 1980).

Table 1 Rock mass classification systems for ground support

<table>
<thead>
<tr>
<th>Classification</th>
<th>Original applications</th>
<th>Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rock load</td>
<td>Tunnels with steel support</td>
<td>Terzaghi (1946)</td>
</tr>
<tr>
<td>Rock quality designation (RQD)</td>
<td>Core logging, tunnels</td>
<td>Deere et al. (1967)</td>
</tr>
<tr>
<td>Rock structure rating</td>
<td>Tunnels with steel support</td>
<td>Wickham et al. (1972)</td>
</tr>
<tr>
<td>Rock mass rating (RMR)</td>
<td>Tunnels</td>
<td>Bieniawski (1973)</td>
</tr>
<tr>
<td>Q classification system</td>
<td>Design of support for underground excavitations</td>
<td>Barton et al. (1974)</td>
</tr>
<tr>
<td>Mining RMR (MRMR)</td>
<td>Support for mining excavations</td>
<td>Laubscher (1977)</td>
</tr>
<tr>
<td>RMi rock classification</td>
<td>Design of support for underground excavitations</td>
<td>Palmstrom (1995)</td>
</tr>
</tbody>
</table>

It is useful to note that the original rock mass classification systems, with the exception of MRMR, were aimed towards civil engineering applications. This was a direct result of the case studies in their respective databases. Although some of these systems have found extensive applications in a mining context, the differences between civil and mining ground support philosophies cannot be underestimated. Civil engineering structures have a longer serviceable life span and are often used by the public, or can contain important and expensive facilities. Mining projects, in general, have a shorter excavation life and access is limited to trained personnel. However, design considerations are often called to account for considerable fluctuations in stress during the life of a project (Choquet & Hadjigeorgiou 1993).

It is recognised that rock support practice has evolved significantly over time. In this respect, since the introduction of RMR and Q, one can observe that new and improved rock reinforcement techniques have been implemented, including many new types of rock bolts, meshing and strapping products and shotcrete reinforcement, including a variety of fibres.

Depending on the classification system, the relation between rock mass quality and support requirements can be based on two approaches. First, as a correlation between a classification index and the ground support pressure requirement, and second, by a simple association between a classification index, a span and a ground support recommendation in the form of bolt spacing and shotcrete thickness; effectively, a support guideline.
2.1 Support pressure

The following briefly summarises the use of an empirical correlation between a classification index, such as RMR and Q, and the resulting support pressure requirement. A weakness of this approach is that the ratings for the constitutive parameters of both classification systems evolved over time. This would suggest that, depending on the version of the classification system employed, for the same ground conditions, this would result in a different rating. This could potentially influence the continued applicability of any empirical correlations between support pressure and classification indices based on earlier versions of the systems.

Unal (1986) proposed the following relationship using RMRₗₙₗ, based on data from US coal mines:

\[ \text{Parch} = \gamma B \frac{(100 - \text{RMR})}{100} \]  

(1)

where \( \gamma \) is the unit weight and \( B \) is the excavation span. Its applicability to hard rock mines, and to different versions of RMR, is not evident.

Barton et al. (1974) provided an empirical equation to determine the support pressure \( P_{\text{arch}} \) (kg/cm²) based on Q and the joint roughness number \( J_r \):

\[ \text{Parch} = \frac{(2.0/J_r) Q - 1/3}{2} \]  

(2)

It was also suggested that a better relation can be obtained by also introducing the number of joint sets \( J_n \), index:

\[ \text{Parch} = \frac{(2J_n^{1/2} Q - 1/3)}{3J_r} \]  

(3)

In effect both equations provide an identical estimate when the rock mass is intersected by three joint sets. It is interesting to note that excavation span is not part of the Barton et al. (1974) equations. Furthermore, the joint roughness number, \( J_r \), and the number of joint sets, \( J_n \), were deliberately assigned a double dependence. Finally, Barton et al. (1974) suggested that the actual Q values be modified if Equations (2) and (3) were to be used to calculate the support pressure along the walls of the excavations.

2.2 Support recommendation guidelines

Support recommendations for each category of rock mass quality can be specific to a particular tunnel span and may or may not explicitly introduce a factor of safety. For example, Bieniawski (1976) provided support guidelines for a 10 m wide horseshoe shaped drill and blast tunnel under 25 MPa of vertical stress. The support recommendations were limited to 20 mm diameter fully grouted rock bolts, shotcrete and steel sets. It is noted that these remain the only guidelines specific to RMR for tunnelling and have not been revised since 1976, even though the constitutive parameters of the RMR have evolved over time and a range of reinforcement and support tools have been introduced in tunnelling.

Laubscher (1977) introduced a series of adjustments to the RMR system by Bieniawski (1976). These modifications were mostly based on experience associated with chrysotile asbestos mining in southern Africa. The RMR was adjusted to account for the effects of changes in stress, influence of blasting and weathering. A series of support recommendations, using both RMR and the adjusted ratings, were provided for mining operations with “stress level less than 30 MPa”. For a variety of reasons, including its origin in cave mining, and the evolution over time in the way the MRMR is estimated, the support recommendations are not widely employed outside cave mining. In fact the Modified RMR of Laubscher (1977) was not identified as a design tool during the benchmarking study for this project (refer to Section 3).

Barton et al. (1974) provided a more comprehensive series of ground support guidelines which were revised by Grimstad et al. (1986), and Grimstad and Barton (1993). These changes were significant and motivated by the need to better meet the requirements of the Norwegian Method of Tunnelling support techniques. The major change since the 1986 chart was the use of wet steel fibre reinforced shotcrete S(fr), which was not available at the time of the first introduction of the Q-system in 1974.
3 Benchmarking of ground control management plans

Maintaining a Ground Control Management plan (GCMP) at mine sites is best practice and, in some jurisdictions, a requirement. A GCMP outlines, and in certain cases defines, the responsibilities, the data collection and design processes of ground control and the frequency of quality control and assurance measures. An integral part of a GCMP are “ground control standards” that define the choice of reinforcement and surface support (bolt type, reinforcement pattern, shotcrete or mesh, etc.) for specific ground conditions. The ground conditions are often delineated by zoning (or domains) within the mine geomechanical model. The ground control standards at hard rock mines are generally the result of an evolution, starting with the feasibility recommendations and adapting them to changes in ground conditions whilst accounting for what has worked best at that mine over time. The implemented ground support systems are therefore compatible with ground behaviour and installing equipment capability and dimensions with respect to excavation size.

As part of the Australian Centre for Geomechanics’ “Ground Support Systems Optimisation” (GSSO) project a comprehensive benchmarking review was undertaken of GCMPs, implemented mainly in Australian and Canadian hard rock underground mines. This revealed that the great majority of mines rely on empirical classification tools for determining preliminary support recommendations, with over 75 per cent of the mines relying on a version of the Q-system and, more specifically, on the Grimstad and Barton chart reproduced in Figure 1.

![Rock mass classification — permanent support recommendation based on Q and NMT. Note the extensive use of S(fr) as permanent support (after Grimstad & Barton 1993)](image_url)

Reinforcement categories:

1) Unsupported
2) Spot bolting, sb
3) Systematic bolting, B
4) Systematic bolting (and unreinforced shotcrete, 4-10 cm), B(+S)
5) Fibre reinforced shotcrete and bolting, 5-9 cm, Sfr+B
6) Fibre reinforced shotcrete and bolting, 9-12 cm, Sfr+B
7) Fibre reinforced shotcrete and bolting, 12-15 cm, Sfr+B
8) Fibre reinforced shotcrete, > 15 cm, reinforced ribs of shotcrete and bolting, Sfr, RR+B
9) Cast concrete lining, CCA
Potvin and Hadjigeorgiou (2015) discussed some “disconnect” between the method, as it was intended to be used by Grimstad and Barton (1993), and how it has been implemented in the mining industry. The extensive review of GCMPs from Australia and Canada has shown that most mines have not adopted the new SRF93 guidelines proposed by Grimstad and Barton (1993) for massive and highly stressed rock mass and instead are still using the original SRF74 estimation, in combination with the guidelines, to guide their selection of ground support systems. This is an interesting finding since the SRF93 changes were specifically motivated to fit this support design chart (Figure 1). This is further discussed in Section 4.6 of this paper.

It is also worth noting that the actual support system implemented at mine sites often diverts significantly from the design recommendations from the above methods. This is not surprising given the origin of the support recommendations of the Q-system is closely aligned to civil tunnelling as opposed to mining drives. These differences can be linked to both to the support philosophy and choice of support systems.

4 Ground control recommendations based on rock mass classifications for mining

4.1 Choice of rock mass classification system

All rock mass classification systems have inherent limitations that are sometimes acknowledged or at times ignored. Irrespective of their limitations there is a need at the early stages of mining to gain an understanding of the spatial variability of rock mass quality and develop domains. This is a requirement for the selection of stabilisation strategies including choice of ground support. It is recognised that for any guidelines to be of use and gain acceptability they must be compatible with current practices at mine sites. The benchmarking study indicated the Q-system is by far the most popular system.

4.2 Data collection

The importance of quality data collection as input for the rock mass classification systems cannot be ignored. Hadjigeorgiou (2012) noted that despite the availability of standards for geomechanical data collection, these are not necessarily applied or enforced at several mining operations. The reasons for this diversity of approach, or non-compliance to site specific standards, have to do with the inherent material variability, economics and corporate culture. The practical issues associated with data collection have been discussed by Milne and Hadjigeorgiou (2000), Palmstrom and Broch (2006), amongst others. Barton (2002) has made the case for histogram logging of each Q-parameter as a means of visualising the inherent variability of the rock mass.

4.3 Implementation of rock mass classification systems

Although in theory the implementation of the collected geomechanical data should be straightforward, there are considerable challenges. Hadjigeorgiou (2012) discussed the role of human resources in data collection suggesting that the process is dependent on employing and training qualified personnel. Unless the people conducting the geomechanical data collection realise how important their effort is and how useful it is to the operation, they do not assign it with the same level of priority as other tasks. This is often the case at the green field stage, where the data collection process is driven by exploration. It is imperative to establish a strategy on who collects data, how qualified they are, how the information is recorded, what are the quality control measures in place, etc.

A common source of error that was observed during the benchmarking is the inconsistent use of the various rock characterisation and classification systems. This is partially driven by the availability of various versions of the different classification systems, whereby some of the parameters and their ratings changes back and forth, resulting in questionable rock mass classification ratings and inaccurate subsequent support recommendations.
A problem particular to the use of the Q-system in mining is a consequence of the popularity of the stability graph method as a tool for open stope design. The stability graph method relies on the use of a modified version of the Q-system based on some adjustments to assign a rock mass stability index (Potvin 1988). In the stability graph method, SRF is not used as it is substituted by a stress adjustment factor resulting in a ‘modified’ Q’ value. At certain mines, however, it appears that Q and Q’ are used interchangeably for the design of support. This is misleading since the Grimstad and Barton chart was based on Q and not Q’. If one uses Q’ instead of Q, the effect of stress (or depth below surface) on ground support requirements, which can be very significant, will be ignored.

4.4 Scope of ground support recommendations

The majority of rock mass classification systems have their origin as tools for the design of ground support for underground excavations. The rock mass quality index was later linked to further applications ranging from estimations of rock mass strength, modulus of elasticity, open stope dimension, cavability of caving mines, slope stability etc.

In the present study the objective is to provide support guidelines for mining drives at the early stages of mine design. There are no claims that these are applicable to other situations. These recommendations are derived based on a review of GCMPs linking ground conditions to ground support standards at 45 mines, from which 148 data points have been extracted. This database is quite different from the support recommendations of the Grimstad and Barton (1993) charts for massive and highly stressed rock masses “…….on the basis of 1,050 new cases from main road tunnels, and to some extent, on the basis of 440 new cases from hydropower tunnels”.

4.5 Excavation span

The excavation span is an integral part of most guidelines for assessing the stability of an excavation. In the original Q-system the concept of the excavation support ratio (ESR) was used to reduce the effective span to reflect the type of infrastructure. In this way the degree of safety and support demanded by an excavation is influenced by the purpose of the excavation, the presence of machinery, the exposure of personnel and/or the public, etc. ESR is allowed to vary from 0.5 for underground nuclear power stations to a range between 2 to 5 for “temporary mine openings”. “Permanent mine openings” were assigned to a range of values from 1.6 to 2.0. There are two important issues to note here. The first one is that the Q-database had a very limited set of case studies from mines. The second observation is that the choice of an appropriate ESR at mine sites appears to be subjective and applied inconsistently. This is not surprising given that having a different ESR rating leading to a different degree of ground support conservatism for temporary and permanent mine opening does not adequately reflect current practices in mines. Wherever mine personnel are exposed, whether it is a permanent or temporary drive, the risk of rockfall must be minimised and the same degree of conservatism in ground support design is generally applied. The differentiation is rather seen in the selection of reinforcement products, which need to be more corrosion resistant for permanent openings.

As the scope of this study was limited to provide guidelines specifically for mining drives, the need for an ESR to distinguish between different sets of excavations becomes unnecessary. The distinction between temporary and permanent mine openings also appears to have led to some confusion. Consequently the ground support guidelines proposed in Section 5 will be specific to mining drives were the range of excavation span is between 4–6 m. The authors do not encourage, or advocate, the use of these guidelines beyond the mining database in hard rock mines.

4.6 Ground support recommendations for extreme conditions

Extreme ground behaviour such as rockburst prone and squeezing ground conditions are encountered in a range of projects both in civil and mining applications. These ground conditions are difficult to capture by any classification system. The Q-system attempts to account for these conditions by using the Stress
Reduction Factor (SRF). In the Q-system there are four groups within the SRF ratings: (a) weakness zones causing loosening or fall-out, (b) rock stress problems in competent rock, (c) squeezing (flow of incompetent rock), and (d) swelling (chemical effect due to water uptake). In the 1974 database the large majority of case studies were in groups (a) and (b), having moderate stress in essentially competent rock (unconfined compression strength/major principal stress) in the range of 10 to 200. Squeezing or swelling problems were encountered in only nine of the 212 case records.

The 1993 version of the Q-system was also characterised by changes to the stress reduction factor (referred to in this paper as SRF\textsubscript{93}) ratings. The original SRF\textsubscript{74} recommendations were found to be insufficient for high stress and massive rock mass:

“The updating of the Q-system has shown that in most extreme cases of high stress and hard massive (unjointed) rock, the maximum SRF-value has to be increased from 20 to 400 in order to give a Q-value which correlates with the modern rock support.....” (Grimstad & Barton 1993)

As massive rock masses affected by high stress conditions are difficult to assess by visual observations, the approach suggests using $\sigma_c/\sigma_1$ to determine SRF\textsubscript{93} in these conditions, which are often prone to rockbursting. According to the latest guidelines, reproduced in Table 2, a low $\sigma_c/\sigma_1$ translates into a high SRF\textsubscript{93} value.

Table 2 The updated SRF\textsubscript{93} table from Grimstad and Barton (1993) with ratings compared to SRF\textsubscript{74}

<table>
<thead>
<tr>
<th>Stress level</th>
<th>$\sigma_c/\sigma_1$</th>
<th>$\sigma_0/\sigma_c$</th>
<th>SRF\textsubscript{74}</th>
<th>SRF\textsubscript{93}</th>
</tr>
</thead>
<tbody>
<tr>
<td>Low stress, near surface, open joints</td>
<td>&gt;200</td>
<td>&lt;0.01</td>
<td>2.5</td>
<td>2.5</td>
</tr>
<tr>
<td>Medium stress, favourable stress condition</td>
<td>200-10</td>
<td>0.01-0.3</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td>High stress, very tight structure. Usually favourable to stability, may be unfavourable for wall stability</td>
<td>10-5</td>
<td>0.3-0.4</td>
<td>0.5-2</td>
<td>0.5-2</td>
</tr>
<tr>
<td>Moderate slabbing after &gt;1 hour in massive rock</td>
<td>5-3</td>
<td>0.5-0.65</td>
<td>5-9</td>
<td>5-50</td>
</tr>
<tr>
<td>Slabbing and rockburst after a few minutes in massive rock</td>
<td>3-2</td>
<td>0.65-1</td>
<td>9-15</td>
<td>50-200</td>
</tr>
<tr>
<td>Heavy rockburst (strain-burst) and immediate dynamic deformations in massive rock</td>
<td>&lt;2</td>
<td>&gt;1</td>
<td>15-20</td>
<td>200-400</td>
</tr>
</tbody>
</table>

The extensive review of GCMPs from Australia and Canada has shown that most mines have not adopted the new SRF\textsubscript{93} guidelines proposed by Grimstad and Barton (1993) for massive and highly stressed rock mass (Table 2). They use the original SRF\textsubscript{74} estimation, in combination with Figure 1, to guide their selection of ground support systems.

Perhaps the following observations can contribute to explain the difficulties in using SRF\textsubscript{93} (Table 2) in mining applications. It is the authors’ experience that rockbursting conditions in deep mines are often the result of a combination of stiff and strong rock (often high $\sigma_c$) and high stress (high $\sigma_1$) and as such, a low $\sigma_c/\sigma_1$ is not always a good indicator of rockbursting conditions. Furthermore, the very large change proposed in the SRF\textsubscript{93} ratings, from a maximum of 20 to 400, contributes to a drastic change in the assessment of the Q-value, in high stress conditions, from previous practices. The visual appearance of a poor rock mass with, for example, a Q of 2 (with a SRF\textsubscript{74} of 20 in the old system) is the same as a Q of 0.1 (with a SRF\textsubscript{93} of 400 in the new system).

Previous work by Peck (2000) and Peck and Lee (2007) tends to corroborate the above observations related to deep mining and the 1993 changes in the determination of SRF. However, the SRF issue must be
addressed, especially for rockbursting and squeezing conditions. A factor to account for stresses within the Q-system, which is representative of deep mines, is necessary in order to reflect the increase in ground support requirement in high stress environments. The original SRF74 was deemed inadequate for high stress according to Grimstad and Barton (1993), but the new SRF93 has not been adopted by industry. This could be due to a lack of awareness or a perception that it is inappropriate. There is strong empirical evidence that in its present form it may not be representative of rockbursting conditions in deep mines.

5 New ground support design guidelines for mining drives

5.1 The database

The GCMPs are a valuable source of data from which new guidelines, well adapted to mining practices, could be developed. This is because GCMPs contain extensive information about ground conditions and ground support. This information is constantly updated as knowledge of the ground condition evolves and ground support practices are optimised. In certain jurisdictions, and in certain mining companies, it is a requirement that the GCMP is reviewed on an annual basis. Therefore, it is reasonable to assume that the ground support standards documented in the GCMPs are a good reflection of successful ground support practices for the ground conditions in which they are applied. The widely used empirical approach based on the Q-system and the many limitations in applying it to mining conditions emphasises the need for improving the method. Based on its popularity, the Q-system still appears to be the most appropriate of the rock mass classification systems for the preliminary design of support. The challenge is to improve the database for mining purposes and using rock reinforcement and surface support systems shown to be successful in stabilising mine drives as opposed to civil engineering tunnels.

5.2 Ground support design variables

The main design decisions for mine drives’ ground support generally include:

- The reinforcement pattern which can be expressed as a bolt density (bolts/m²).
- The type of surface support (mesh versus reinforced shotcrete). Note that reinforced shotcrete can include either fibre or mesh reinforcement. Sometimes mesh is also installed over fibre reinforced shotcrete. Plain shotcrete is not often used in mines.
- The thickness of shotcrete, when reinforced shotcrete is selected.
- The coverage of the ground support down the wall; whether the last row of bolts stops at the shoulder of the drive (generally more than 3 m from the floor), around mid-height (1 to 3 m from the floor) or is taken down to within a meter of the floor.

Therefore, these design decisions will be the focus of the new proposed mining guidelines for ground support design. The above four variables were extracted from ground support standards from 45 mines’ GCMPs collected during the benchmarking study and correlated to the Q-value (using SRF74). The data is graphically displayed in Figures 2 to 5 with the following explanations.

In Figure 2 the bolt density is colour-coded with high density bolting in hot colours and low density in cold colours, and plotted on a Q- versus span graph. The graph contains 141 points representing ground support standards. It should be emphasised that the ground support standards are applied to multiple drives that are in the same geomechanical domain and, therefore, the ‘real’ database behind these graphs is significantly more extensive than the 141 case studies. The variation in span in the database is not significant but the data clearly shows a trend for better quality rock mass, and a lower bolt density has been used (cold colours to the right side of the graph). Four zones have been defined on the graph; $Q < 1.0$, $1.0 < Q < 4.0$, $4.0 < Q < 10.0$, $Q > 10.0$, following Barton et al. (1974) classification of extremely and very poor, poor, fair, and good ground.
Figure 2 Bolt density as a function of rock mass quality Q

In Figure 3, the type of surface support is indicated using three different shapes of markers: round markers represent fibre reinforced shotcrete, square represent mesh, and triangular markers are cases where shotcrete and mesh were used together, in most cases, the mesh was installed externally, on top of fibre reinforced shotcrete. The graph in Figure 3 also contains 141 ground support standard observations. It is observed that for excavation spans smaller than 5 m, reinforced shotcrete has not frequently been used in the database. When Q is smaller than 1.0, the majority of cases (65 per cent) have used reinforced shotcrete or a combination of reinforced shotcrete and mesh. When Q is greater than 10.0, the majority (86 per cent) of cases have used mesh only. When Q is between 1 and 10, productivity considerations rather than ground conditions alone may have a strong influence on the selection of mesh versus reinforced shotcrete. As a generalisation, mesh seems to provide adequate support for ground conditions where Q > 1.0, albeit with a higher bolt density than when reinforced shotcrete is used.
Selection of ground support for mining drives based on the Q-System

Y. Potvin and J. Hadjigeorgiou

Figure 3  Reinforced shotcrete versus mesh as a function of rock mass quality Q

Looking further into the data and combining information from Figures 2 and 3, it is possible to calculate the average bolt density and standard deviation used with mesh versus fibre reinforced shotcrete for each rock mass classification category. The bolt density data combined with surface support is summarised in Table 3.

Table 3  Bolt density (with standard deviation provided in bracket) used with mesh versus reinforced shotcrete as a function of Q

<table>
<thead>
<tr>
<th>Bolt density with mesh (bolts/m²)</th>
<th>Q &lt; 1.0</th>
<th>1.0 &lt; Q &lt; 4.0</th>
<th>4.0 &lt; Q &lt; 10.0</th>
<th>Q &gt; 10.0</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.77* (0.26)</td>
<td>0.72 (0.20)</td>
<td>0.65 (0.11)</td>
<td>0.56 (0.14)</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Bolt density with fibre reinforced shotcrete (bolts/m²)</th>
<th>Q &lt; 1.0</th>
<th>1.0 &lt; Q &lt; 4.0</th>
<th>4.0 &lt; Q &lt; 10.0</th>
<th>Q &gt; 10.0</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.66 (0.20)</td>
<td>0.51 (0.07)</td>
<td>0.47 (0.05)</td>
<td>0.40 (0.06)</td>
<td></td>
</tr>
</tbody>
</table>

*Looking at the general trend, the bolt density with mesh when Q < 1.0 appears to be low. This is believed to be due to the fact that more than half the cases used in the calculation had span smaller than 5 m. If one considers only spans greater than 5 m, the bolt density increases to 0.87. This later value will be used for the purpose of developing the guidelines.

Figure 4 displays shotcrete thickness variations as a function of Q. The graph contains 57 observations. Despite the small database, the general trend of using thicker reinforced shotcrete layers in poorer ground is observed. In most cases where Q was greater than 1, a 50 mm thickness was employed. When Q was smaller than 1.0, either 75 or 100 mm shotcrete thickness was applied. Although the data is still somewhat scarce, 100 mm layers tend to be used when Q < 0.2.
Figure 4 Reinforced shotcrete thickness as a function of rock mass quality Q

Figure 5 shows the extent of reinforcement and surface support coverage applied to walls, expressed as the distance between ground support and the floor (or the height of the unsupported wall). Three categories are defined:

1. Floor: when the ground support extends down to within 1 m of the floor.
2. Mid-drift: when the ground support stops around the mid-height of the drive, 1 to 3 m from the floor.
3. Shoulder: when the ground support extends to more than 3 m from the floor.

When Q is smaller than 1.0, the reinforcement and surface support coverage is often extended to near the floor. In poor ground (1.0 < Q < 4.0) the majority of cases show the ground support stopping around mid-drift. When ground conditions are fair or good (Q > 4.0), it is more common to have the wall support only reaching the shoulder of the drive, leaving about 3 m or more of unsupported wall height.
5.3 Ground support guidelines for mining drives

The proposed guidelines for ground support in mining drives derived from the data displayed in Section 5.2 have been compiled and rounded-up to produce the guidelines shown in Figure 6. These guidelines provide recommendations in the preliminary design of reinforcement and support for mining drives. They are supported by GCMP data from 45 mines, and as such they are based on the experience of successful reinforcement and on the geomechanical domains based on the Q. Its applicability is for mining drives of 4 to 6 m span and therefore eliminates the subjectivity of selecting a specific ESR for other types of excavations.
It is noted that these are general guidelines that are meant to be used as a first pass design at the early stages of mine life (pre-feasibility, feasibility studies and early mine development). The guidelines reflect safe practices documented in the GCMPs of many Australian and Canadian mines. The support design is likely to be refined as experience in local ground conditions is gained.

There is a wide range of reinforcement products with different load and displacement capacity available on the market. Although most of the products would be represented in the database used to develop the proposed guidelines, no specific recommendations on bolt types are provided. Specifying a particular bolt with a certain capacity is not seen as being critical because the guidelines rely on the principle of reinforcing the rock mass rather than suspending the dead weight of failed rock. As such the bolt density required to 'lock-in' the rock mass overrides the need for a defined bolt capacity. The underlying assumption is that the vast majority of bolts in a support system are not likely to be loaded to their full capacity but are meant to minimise initial displacement of the rock mass and avoid rock mass loosening. The same principle applies for the various fibre reinforced shotcrete or mesh products. This assumption is supported by the variety of support products contained in the successful bolting patterns documented in GCMPs and implies that most “proven” commercial bolts can be safely used with the recommended bolting density. It is also noted that long term excavations tend to use fully grouted bolts to provide extra protection against corrosion.

The following describes important characteristics of the database that users of the guidelines need to be aware of. Ground support practices are strongly influenced by the culture and practices of mining regions. In Australia, mesh installation is generally performed at the same time as primary bolting using a jumbo drill, relatively large sheets 2.4 x 3.0 m are often installed. This facilitates high productivity with a row of six sheets covering up to 40 m², accounting for a 3 square overlap between the sheets. This has shown to have a significant influence on bolt density. The default standard specification of wire is 5.6 mm diameter and the aperture is generally 100 x 100 mm. Such a sheet weighs approximately 25 kg.

In North America, where the mesh installation involves more manual handling, either with air-leg or Mclean bolter, smaller and lighter sheets are often used. For example, common sheet dimensions are 1.2-1.5 x
3.0 m. The wire also varies with the three most common diameters being 3.8 mm (# 9 gauge), 4.9 mm (#6 gauge) and 5.8 mm (# 9 gauge). Like in Australia, the standard aperture used is 100 × 100 mm.

When a guideline refers to shotcrete, it implies reinforced shotcrete and excludes plain shotcrete. The type of reinforcement, whether it is mesh or fibre (steel and synthetic), is not specified as all types are acceptable and represented in the database. The Canadian mines tend to use dry shotcrete, often mesh reinforced, whilst the Australian mines generally use wet fibre reinforced shotcrete.

5.4 The limitations of the ground support guidelines

The proposed guidelines are not valid for either squeezing ground or rockburst prone conditions. In essence, the principle of pattern bolting to lock-in the rock mass is not relevant to these ‘extreme’ failure mechanisms.

Dynamic or yielding bolts are increasingly popular in deep and high stress mines and a number of new products have been made available on the market during the last decade. At this stage, there is not enough empirical data to include dynamic support in new empirical guidelines. Furthermore, it is unlikely that a simple relationship between span and Q would allow deriving guidelines from such a complex problem as supporting rock mass subjected to dynamic loading, especially given the unclear situation with SRF described earlier. Parameters related to the magnitude of the largest possible seismic events would necessarily need to be accounted for. Therefore, dynamic ground support is not included in the proposed guidelines. Similarly, squeezing ground requires guidelines of its own and is not implicitly included in this study.

6 Conclusion

New empirical ground support design guidelines for mine drives have been developed based on GCMPs, as they contain extensive and updated information on ground conditions and ground support. The need for new guidelines was driven by the realisation that the Grimstad and Barton (1993) chart was extensively used in the mining industry, and that there are many limitations, either ignored or previously unidentified by the users. The popularity of this design graph is largely due to there not being any other practical alternatives for empirical evaluation of support designs at the early stages of mining.

As for the limitations of the popular Grimstad and Barton (1993) graph, firstly the database used to build the graph is entirely from civil engineering, and secondly the guidelines have been developed based on a large scatter of bolting patterns, as pointed out by Palmstrom & Broch (2006). More importantly, it is self-evident that the many differences between civil tunnelling and mining cannot be embodied into the ESR factor alone.

The changes between SRF74 and SRF93, which were meant to better represent massive rock under high stress conditions, have been ignored by the mining industry, despite the fact that the design chart is meant to be applied with SRF93. Technically, since this is an empirical system, it is not appropriate to extrapolate from civil to mining practices and, furthermore, to use factors which are different (SRF74) than the ones used for developing the chart (SRF93).

The new guidelines address most of the above issues and provide a practical tool for mine designers to assess ground support requirements empirically at the early stages of mine design. The key design parameters include: Bolt density, the use of reinforced shotcrete versus mesh. The thickness of reinforced shotcrete if selected. Reinforcement wall coverage — to install reinforcement to gradeline (3.5 m from the floor) or close to the floor (approximately 1 m). Surface support wall coverage — to install surface support to gradeline (3.5 m from the floor) or close to the floor.

The guidelines are seen as a conservative first pass design to be implemented until experience is gained. The main characteristics and ground support practices behind the database has been briefly summarised in Section 5.3 and prospective users of the guidelines are urged to familiarise themselves with those
characteristics to ensure that their design stays within the empirical boundaries of the database. The proposed empirical guidelines are only one of the possible approaches to be used for the design of support.

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Special thanks are extended to Ruth Stephenson and AMC Consultants for providing a significant number of case studies where the mine owners’ names were kept confidential; and to Daniel Cumming-Potvin who performed most of the data manipulation in mXrap and provided input into the analysis.

We are grateful to a significant number of individuals and mining operations in Canada and Australia outside of the GSSO sponsors group that have assisted by providing data.

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Sensor techniques to monitor installation and status of rock bolts

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1 Abstract

1.1 Sensor techniques to monitor installation and status of rock bolts

As today’s mining goes deeper than ever before and the in-situ stresses increase, rock reinforcements become increasingly more important to maintain safe and stable underground constructions. There is a great demand to know if the rock reinforcement performs as designed. For example, it can be difficult for a rock engineer to know if the rock bolts are fully grouted, if the rock bolt has broken inside the rock mass or how much elongation has occurred along a dynamic rock bolt inside the rock. To handle these problems, two techniques have been developed.

1.1.1 Quality assurance of the installation

To measure how well the rock bolt is grouted in a borehole, the bolt has been fitted with a plastic tube with small cuts along the length of the bolt. After the bolt is installed, the plastic tube is pressurized to detect cavities and cracks. If there are cavities in the grout, the gas will pass through the plastic tube and fill the cavities, making it possible to calculate their size.

A system called Cavimeter has been developed using this technique, and has been tested on different types of rock bolts. More than 1000 measurements have been made in different types of rock mass at various sites. The results show that the quality of the bolt installation can be investigated using this technique. The Cavimeter is available on the market today.

1.1.2 Monitoring the status of installed rock bolts

Another technique has been developed to monitor for example if a dynamic rock bolt has been broken or if the bolt has been elongated inside the rock mass. A small steel wire is put inside the plastic tube and is fastened at the toe of the rock bolt. By measuring the current loop through the rock bolt and back through the steel wire, the status of broken or not can be determined. Elongation of the rock bolt can be determined by measuring the offset between the thread and the steel wire.

Laboratory tests and preliminary field test show that this technique works. Two types of sensors have been tested. One gives warnings with a blinking red LED if the rock bolt is broken inside the rock and a blinking yellow LED if the elongation of the rock bolt is more than the accepted length. The other sensor can communicate wirelessly with the network of the mine. The data can be displayed on a central computer.

2 Background

As today’s mining goes deeper than ever before and the in-situ stresses increase, rock reinforcements become increasingly more important to maintain safe and stable underground constructions. There is a great demand to know if the rock reinforcement is performing as designed. For example, it can be difficult for a rock engineer to know if the rock bolts are fully grouted, if the rock bolt has broken inside the rock mass or how much elongation has occurred along a dynamic rock bolt inside the rock. To handle these problems two techniques have been developed.

One is to quality-assure the installation of rock bolts by using the Cavimeter method and the other is the Cavisensor program to monitor the elongation and breakage of the installed rock bolts.
3 The Cavimeter method

The rock mass can be reinforced by using rock bolts. Rock reinforcement by means of cement-grouted rock bolts starts by drilling a hole in the rock. Cement slurry is then pumped into the borehole and a rock bolt is inserted. The bolt can be anchored in the bottom of the borehole, using a wedge or expander, before it is tensioned to a certain torque.

In order to be fully functional, the rock bolt must be adequately grouted. If the cement slurry runs out into cracks or cavities in the rock mass, the reinforcement may not carry the intended load. If the borehole is not fully filled with concrete, there is also a risk for corrosion problems because of water and air coming in contact with the steel of the rock bolt.

To measure how well the rock bolt is grouted, the bolt is fitted with a plastic tube with small cuts along the length of the rock bolt. After the bolt is installed, the plastic tube is pressurized to detect cavities and cracks. If there are cavities in the grout, the gas will pass through the plastic tube and fill the cavities, making it possible to calculate their size.

Figure 1 Principle of the Cavimeter system

A system called Cavimeter, see Fig. 1, has been developed using this technique. It has been tested on different types of rock bolts and wire bolts.

3.1 Field measurements with the Cavimeter

More than 1000 measurements have been made as of today in different types of rock mass at various sites, see Table 1. In Table 1, the size of the cavities are not listed, so they may vary in size.

Table 1 Statistics of measurements 2010 – 2013 with the Cavimeter

<table>
<thead>
<tr>
<th>Year</th>
<th>Type of industry</th>
<th>Type of rock bolts</th>
<th>Percentage of rock bolts where cavities were detected</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>2010</td>
<td>Mining</td>
<td>30 static rebar</td>
<td>17% with cavities</td>
<td>Test</td>
</tr>
<tr>
<td>2010</td>
<td>Infrastructure</td>
<td>10 static rebar</td>
<td>20% with cavities</td>
<td>Two were badly grouted on purpose</td>
</tr>
<tr>
<td>2010</td>
<td>Mining</td>
<td>132 static rebar</td>
<td>9% with cavities</td>
<td>Good rock mass</td>
</tr>
<tr>
<td>2010</td>
<td>Mining</td>
<td>235 static rebar</td>
<td>7% with cavities</td>
<td>Good rock mass</td>
</tr>
<tr>
<td>2011</td>
<td>Mining</td>
<td>50 static rebar</td>
<td>33% with cavities</td>
<td></td>
</tr>
<tr>
<td>2011</td>
<td>Infrastructure</td>
<td>20 static rebar</td>
<td>40% with cavities</td>
<td>Tunnel</td>
</tr>
<tr>
<td>2011</td>
<td>Mining</td>
<td>150 dynamic bolts</td>
<td>27% with cavities</td>
<td></td>
</tr>
<tr>
<td>2011</td>
<td>Mining</td>
<td>150 dynamic bolts</td>
<td>15% with cavities</td>
<td>water</td>
</tr>
</tbody>
</table>
When measurements are made today with the Cavimeter, the filling ratio in % is shown on the handheld presentation unit directly after the measurement is made, see Fig. 2. This data is not noted in Table 1 because this function was implemented after 2013.

![Figure 2](image)

**Figure 2** Filling ratio measured with the Cavimeter on a plastic tube to simulate a not fully grouted rock bolt.

The quality of the rock bolt installation can be checked by using this technique. Today the Cavimeter method is patented and the Cavimeter is on the market.

### 3.2 The theory behind the Cavimeter

Static as well as a dynamic mathematic consideration on how the Cavimeter measuring method works are presented here. The static approach is represented with simple calculations.

The mathematical model developed in the dynamic approach are based on Laplace transforms. The mathematical tool used assumes that the system is regarded as linear and that the system is at rest at time $t = 0$. An engineering approach indicates that this approximation is acceptable.

#### 3.2.1 Static approach

Below are some simplified calculations of cavity sizes. Note that only the initial values and final values (typically after 5 seconds) have been taken into account in this simplified calculation.

**Definitions:**

- $P_s$ = Initial pressure in the gas cylinder inside the Cavimeter
- $V_s$ = Volume of the gas cylinder inside the Cavimeter
- $P_f$ = Final pressure in the gas cylinder inside the Cavimeter
- $V_{tot}$ = Total volume consisting of the gas cylinder inside the Cavimeter, plastic tubes and possible cavity
- $V_c$ = Volume of the cavity in the cement grout
- $V_t$ = Volume of the plastic tubes

The temperature is presumed constant:

$$P_s V_{tot} = P_f V_s$$

$$V_{tot} = V_s + V_t + V_c$$
\[ V_c = \frac{P_s}{P_e} V_s - V_t = V_s \left( \frac{P_s}{P_e} - 1 \right) - V_t \]

\( V_s = 1 \text{ dm}^3 \) and the dimension of the plastic tube, e.g. \( V_s \), is known. Then if \( P_s \) and \( P_e \) are observed the size of possible cavities can be calculated.

### 3.2.2 Dynamic approach

Two cases are illustrated below. The first case indicates the impact of a present cavity in the cement grout. The second case indicates the impact of a present cavity as well as micro cracks in the cement grout and rock.

Some simplifications have been assumed:

The system is linear and the starting point is at \( t=0 \). We regard the gas cylinder inside the Cavimeter as a capacitor \( C_1 \) with initial pressure \( P_0 \) and the cavity in the cement grout as a capacitor \( C_2 \) not pressurized at \( t=0 \).

### 3.2.3 Effect of a present cavity

Laplace transform (s operator) is used to solve the differential equation represented by Fig. 3.

Using the inverse Laplace transform to obtain a time dependent equation on how the pressure in the gas cylinder inside the Cavimeter varies:

\[ p(t) \]

Initial pressure: \( P_0 \)

Asymptote: \( P_{\infty} \)

Graphs are plotted for cavities 1, 2, 3, 4 and 5 times the volume of the gas cylinder inside the Cavimeter with initial pressure 3 bar, see Fig. 3.

Practical tests using the Cavimeter on known voids 1, 2, 3, 4 and 5 dl demonstrate almost identical results, see Fig. 4.
3.2.4 Effect of a present cavity and cracks in the cement grout and rock mass

A resistor is used to model cracks in the cement grout. The smaller the cracks are, the greater the resistance, see Fig. 5.

Using the same mathematic method on the model represented in Fig. 7 we obtain:

\[
\text{Where } s_1 \text{ and } s_2 \text{ are computable real roots of the denominator of the Laplace transform.}
\]

Graphs are plotted for cavities with the same volume as the gas container inside the Cavimeter and different numbers of micro cracks, see Fig. 5. Initial pressure is 3 bar.

In order to verify the theoretical simulation, practical tests with the Cavimeter were carried out. An adjustable valve was used to simulate different numbers of micro cracks in the cement grouting and rock mass. The valve was opened step-by-step and measurements were made, see Fig. 6.
Measurements were made with the Cavimeter using different settings of the valve. The valve was adjusted to simulate different numbers of micro cracks. Screen dump of the Cavimeter interface after finished measurements produce almost identical curves shapes.

3.2.5 Conclusion

If the graph of the Cavimeter output slopes downward and then levels out, there are cavities present. The change in pressure value is proportional to the volume of the cavities. The more the pressure falls, the bigger the cavities. If the graph slopes but does not level out, there are micro cracks present, see Fig. 7.

---

**Figure 6** Measurements were made with the Cavimeter using different settings of the valve. The valve was adjusted to simulate different numbers of micro cracks. Screen dump of the Cavimeter interface after finished measurements produce almost identical curves shapes.

**Figure 7** The principle of how the Cavimeter graphs can be interpreted. In the figure orange = cement grout and green = gas penetrating cavities and micro cracks.
To measure one rock bolt normally don’t take more than 5 seconds. The built in evaluating software will display filling ratio as a percentage of a fully grouted bore hole, on the handheld presentation unit, as seen in Figure 2.

If there are micro cracks present in the cement grout and rock mass the Cavimeter will display “LEAK” instead of a percent number. The input parameters to calculate this are: Bore hole diameter, bore hole length, rock bolt diameter and the length and diameter of the plastic tube between the Cavimeter and the toe of the rock bolt. This data could be typed in using the handheld presentation unit.

4 The Cavisensor program

The development of the patented Cavisensor program started in 2014. One objective was to monitor the elongation of installed dynamic rock bolts. Another objective was to monitor when a rock bolt breaks inside the rock mass. The lineup of the sensor program was:

CaviBasic  a simple extensometer mounted on a rock bolt. It consists of a plastic tube with a steel wire inside. The steel wire is fastened at the toe of the rock bolt and is free to move inside the plastic tube. Measurement of elongation and bolt breakage are made by hand using an ohm-meter and a digital caliper.

If the plastic tube is prepared with small cuts along the length of the tube it is possible to use the Cavimeter to check the quality of the rock bolt installation.

CaviLight  a sensor screwed on to the thread of a rock bolt. The sensor can indicate bolt breakage and elongation of installed rock bolts. CaviLight is based on CaviBasic and requires that the rock bolt is installed with CaviBasic. Bolt breakage is indicated by a flashing red LED mounted on the sensor. Bolt elongation can be measured by hand or by a fluorescent bead falling down from the sensor when a pre-determined elongation is achieved.

CaviSens  a sensor screwed on to the thread of a rock bolt. The sensor can indicate bolt breakage by a flashing red LED and elongation by a flashing yellow LED if an adjustable pre-determined elongation is achieved. CaviSens is based on CaviBasic and requires that the rock bolt is installed with CaviBasic.

CaviCom  CaviCom is based on CaviSens and is complemented with an electronic board that can communicate wireless with a gateway. The gateway is implemented with Wi-Fi which enables communication with the mine network and thereby presentation of rock bolt status on a central computer.

The full sensor program has been tested in a laboratory environment with great success. CaviCom was tested together with Mobilaris AB’s presentation system MMI (Mobilaris Mining Intelligence) that is used by LKAB and Boliden as well as other mining companies.

4.1 CaviBasic

CaviBasic can be regarded as a simple and low-cost extensometer point welded at the toe of any mounted rock bolt or wire bolt. It consists of a plastic tube with a steel wire (piano wire) inside. The steel wire is fastened at the toe of the rock bolt and is free to move inside the plastic tube. Measurement of elongation and bolt breakage are made by hand using a caliper and an ohm-meter.

A field test of CaviBasic began in July 2015 at Outokumpu’s mine outside Kemi and is still running. Field tests of the other sensors are planned to start in early spring 2016 and are planned to be completed in autumn 2016.
4.1.1 Principle of CaviBasic operation

A small steel wire is put inside a flexible plastic tube and is fastened at the toe of the rock bolt. By measuring the current loop through the rock bolt and back through the steel wire, the status broken or not can be determined. Elongation of the rock bolt can be determined by measuring the offset between the thread and the steel wire, see Fig. 8.

![Figure 8](image1.png)

Figure 8 Principle of CaviBasic operation to display elongation of rock bolts.

4.1.2 Field test at Kemi mine

Thirty (30) dynamic rock bolts of type NMX Dynamic M24 were prepared at Nybergs Mechanical Workshop in Kiruna. The bolts were prepared with a plastic tube Ø_o=4.0 mm and Ø_i=2.5 mm. A piano wire Ø=0.7 mm was inserted into the plastic tube and fastened at the toe of the bolt, see Fig. 9.

![Figure 9](image2.png)

Figure 9 Rock bolts prepared with CaviBasic

The Outokumpu’s mine outside Kemi experiences considerable rock movement in certain areas. The idea is to use CaviBasic to measure the progress of these rock movements. A simple and reliable way to do this is to measure the displacement between the rock bolt thread and the free moving piano wire that is attached at the toe of the rock bolt. Moreover, by measuring the resistance between the rock bolt thread and piano wire it is possible to detect bolt breakage inside the rock.

During installation of Cavibasic, the hole nearest the center of the washer is preferable because the measuring tube and piano wire will be more parallel to the rock bolt than if using the outer hole of the washer, see Fig. 10.
4.2 CaviLight

A sensor screwed on to the thread of a rock bolt equipped with CaviBasic. The sensor can indicate bolt breakage and elongation of installed rock bolts. Bolt breakage is indicated by a flashing red LED mounted on the sensor, see Fig. 12. Rock bolt elongation can be measured by hand and/or indicated by a fluorescent bead falling down from the sensor when a pre-determined elongation is achieved.

The theoretical lifespan of the sensor is 5 years if the red LED is not flashing. If the LED is flashing the lifespan is reduced to approximately one week. The sensor is reusable, completely sealed and can operate in a rough environment. A dedicated flashlight can be used to verify that the sensors are in operation.
The CaviLight sensor has been successfully tested in a laboratory environment and field tests are planned to start in early spring 2016.

4.3 CaviSens

CaviSens is sensor screwed on to the thread of a rock bolt. The sensor can indicate bolt breakage by a flashing red LED and elongation by a flashing yellow LED if an adjustable predetermined elongation are achieved. CaviSens are based on CaviBasic and requires that the rock bolt are installed with CaviBasic.

4.3.1 Principle of CaviSens operation

A small steel wire is put inside a flexible plastic tube and is fastened at the toe of the rock bolt. By measuring the current loop trough the rock bolt and back through the steel wire the status of broken or not can be determined.

Elongation of the rock bolt can be determined by measuring the offset between the thread and the steel wire. A built-in potentiometer will produce a voltage proportional to elongation of the rock bolt. By adjusting another potentiometer inside the sensor housing it is possible to adjust at which elongation the yellow LED should start to flash, see Fig. 13.

It is also possible to connect a data logger to the sensor to log data as elongation and bolt breakage as a function of time. This is done in the Kemi mine during field testing of one CaviSens unit.
4.3.2 Laboratory test of CaviSens

Laboratory tests of CaviSens were carried out at Nybergs Mechanical Workshop in Kiruna in January 2015. Four test objects were prepared, see Fig. 14.

![Diagram A: An 8 mm steel bar was prepared with a 400 mm long plastic tube to create a dynamic zone.](image1)

![Diagram B: The steel bar was cement-grouted inside two separate steel pipes.](image2)

![Diagram C: A reference potentiometer was mounted on the hydraulic jack and fastened on the moving cylinder of the hydraulic jack.](image3)

![Diagram D: The actual setup for the tests. To the left an early prototype of the CaviSens.](image4)

Figure 14 Preparation of laboratory tests of CaviSens. A) An 8 mm steel bar was prepared with a 400 mm long plastic tube to create a dynamic zone. B) The steel bar was cement-grouted inside two separate steel pipes. C) A reference potentiometer was mounted on the hydraulic jack and fastened on the moving cylinder of the hydraulic jack. D) The actual setup for the tests. To the left an early prototype of the CaviSens.

The signals from the test object were amplified and low pass filtered and captured by an AD converter from National Instruments. Data was saved and displayed on a computer. Figure 15 shows the reference signals of load and elongation as well as output elongation and bolt breakage from the CaviSens prototype.

![Diagram E: A diagram from the performed test. The reference potentiometer was protracted approx. 3 cm when the test started.](image5)

4.3.3 Conclusion

The results indicated that measuring the elongation could be done using the technique with a piano wire inside a plastic tube where the wire is fastened at the toe of a rock bolt. It is also possible to indicate if a rock bolt has snapped inside the rock by checking the current loop through the rebar and back through the wire which is isolated by the plastic tube.
4.3.4 Field test of CaviSens at Kemi mine

One CaviSens was also tested together with field tests of CaviBasic in Outokumpu’s mine outside Kemi. Rock bolt number 9 was used for that purpose. Instead of flashing red and yellow LED’s, a contact was mounted on the sensor. From the contact a cable was connected to a data logger placed inside a sealed plastic box, see Fig. 16.

![Figure 16 CaviSens and data logger](image)

To adjust the signal levels of the sensor to the data logger in order to enhance the performance an amplifier and filter were built and mounted inside the sensor housing. The amplifier had to be battery powered. The battery was placed inside a sealed plastic box together with the data logger.

As seen in Table 2, rock bolt number 9 has had an elongation of 0.83 – 0.53 ≈ 0.3 mm as of 2015-11-06.

<table>
<thead>
<tr>
<th>Bolt number</th>
<th>Readout 2015-07-29 (mm)</th>
<th>Readout 2015-10-13 (mm)</th>
<th>Readout 2015-11-06 (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>9 (test of CaviSens)</td>
<td>0.56</td>
<td>0.63</td>
<td>0.83</td>
</tr>
</tbody>
</table>

More field tests are planned to be performed in early spring 2016 at LKAB in Malmberget and SKB (Swedish Nuclear Fuel and Waste Management Co.) in Oskarshamn.

4.4 CaviCom

CaviCom is based on CaviSens and is complemented by an electronic board (internet of things) that can communicate wireless with a gateway. The gateway is implemented with Wi-Fi which enables communication with the mine’s network and thereby presentation of rock bolt status on a central computer, see Fig. 17.
The full sensor program has been tested in a laboratory environment with great success. CaviCom was tested together with Mobilaris AB’s MMI (Mobilaris Mining Intelligence) that are used by LKAB and Boliden as well as other mining companies.

Field tests are planned to start early spring 2016 at LKAB in Kiruna.

5 Acknowledgements

Suggestions and support from Jari Näsi (Outokumpu OY) during tests in Kemi are gratefully acknowledged.

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Strain of steel rebar vs. rock bolt elongation on a laboratory stand

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Abstract

Rock bolts used in deep underground metal ore mines work under static or dynamic loading. This article describes an original laboratory test facility which enables rock bolt testing under static loading conditions. This construction enables testing of different solutions, including expansion shell bolts cemented in a borehole based on resin capsules, long steel and cable or string bolts on a real scale. The maximum axial loading capacity is 1,600 kN. For the experiments under consideration, only 400kN range was used to characterise strain-stress characteristics. Measuring equipment used as well as the possibilities of the test facility were characterized. Tests were conducted on expansion rock bolt supports installed inside a high strength concrete block simulating rock mass with uniaxial compression strength of 80MPa. The rock bolts were loaded statically. The loading programme enabled steady increase of the tension force acting on the bolt. The experimental stress-strain characteristics of the rock bolt show a relationship between strain of the steel rebar and elongation/displacement of the rock bolt, resulting from the deformation of the bolt components (washer, thread part, head). Two specific rates with various levels of intensity of strain/displacement per unit have been distinguished for the rock bolt as such and the rod.

Key words: rock bolt testing, laboratory stand, rock mass reinforcement, measurements methods

1. Introduction

Underground mining, particularly at greater depths, is done under static and dynamic loads. In room and pillar mining methods, pillars are gradually being destroyed, followed by separation of roof rocks in the case of sedimentary and laminated rocks. The lithological diversity, layer divisibility and the presence of weakened surface contact between the layers cause bending of roof layers, cracking and, subsequently, loss of stability of excavations. In the future, we will have to deal with the worsening dynamic effects of the circumstances described above, described by many authors (Korzeniowski et al., 2001; Nierobisz, 2012; Thompson et al., 2012; Cai, 2013). Modern rock bolt support structures are often based on a material with high tensile strength, which, apart from preserving the required load capacity, also ensures acceptable parameters of elongation, axial displacement of the rock bolt or deflection of the supported ceiling (Madziarz, 2002; Korzeniowski & Skrzypkowski, 2011; Skrzypkowski, 2012). In mining excavations reinforced with rock bolt support, there is danger of unforeseen roof rock falls. One of the conditions for safe mining is the ability to maintain certain geometrical parameters of the excavation within the required period. In ore mining there are cases that span smaller than the limits, symptoms of local instability in the form of rock caving or landslides; critical separation of roof occurs due to the effects of structural disturbance (Korzeniowski, 1994, 1999). For many years, both in Poland and around the world, research has been carried out to improve the design of the rock bolt support structure and its interaction with the rock mass, as well as in scope of its validation or development of optimal designing methods. The most reliable research results are obtained where the range of the load and loading conditions correspond to “in situ” environments (Stoiński, 1990). Many research works on the properties of individual components apply to
both the individual rock bolts and support structures and are performed at full scale (Dudzic T., Korzeniowski W., Piechota S., 2004, Radwanska, 2005.)

In the following part, a research methodology and sample results of rock bolt loading, carried out on the laboratory’s original stand, have been analysed in details.

Usually, while analysing rock bolt loading and its properties, we test loading and deformation of bolt. From a mining technology point of view, it is also very important what the range of deformation and displacement of the reinforced rock mass layers within the “bolted zone” is. In many cases in mines (especially in the room and pillar method), one can observe forward and backward displacements of the roof that are higher than acceptable elongation of each single bolt, depending on the properties of the material. For this case we can use a deformable plate instead of a stiff one, which is placed between the bolt nut and the surface of the roof. As a result we get equivalent loading capacity of the bolt with extended elongation of the bolt (thanks to the deformable plate) and of course the reinforced zone of the roof. This is the reason the total displacement measured with the encoder is much bigger as compared with the strain gauge point measurement which corresponds first of all with the steel strain parameters.

The second reason the work has been done is to present the unique laboratory facilities for rock bolting as the basic and common reinforcement method in underground mining.

2. Laboratory test facility for testing rock bolts

The laboratory stand (HUK) for testing the tensile strength of rock bolts designed in the Department of Underground Mining, AGH, enables the testing of rock bolt supports under various loading conditions. It consists of hydraulic loading arrangement of rock bolts and control panel controlled by computer (Korzeniowski W., Skrzypkowski K., Herezy Ł., 2015). Loading is applied to the system by a hydraulic power supply unit constructed on the basis of an axial piston pump. Thanks to the servo-controlled set, a constant oil pressure may be kept at a defined level. The power supply (18kW) is located in a separate room in order to eliminate excessive noise during its operation. Maximum oil pressure in the pump is 31 MPa. An amplifier QUANTUM MX840 with CATMAN-EASY software, connected with loading/strain sensors, is used for necessary strain and forces measurements.

Tests on the rock bolt support can be carried out using the appropriate operating mode of the HUK stand. As a first step the rock bolt should be installed in a borehole within the concrete block. The support installation mode is used to perform the preparatory activities prior to the trial. It served to set all working components in their required positions. With regard to tests on the expansion rock bolt, it is treated as initial tension of the rock bolt. The proper loading mode, whether static or dynamic, is established after fixing the bolt. Loading force is gradually increased in the static mode. The rock bolt may remain under applied force at a defined period of time. In the dynamic load mode, the rock bolt may be stretched and ruptured by the use of maximum force in the shortest possible time resulting from the power and efficiency of the pump.

2.1. Hydraulic loading arrangement of rock bolts—HUK

The HUK stand (Fig. 1) consists of a support frame (1) on which a block of the hydraulic cylinders (2), (3) is placed. They may be combined in pairs of 2, 4 and 8 of servo motors operating together, depending on the strength of the test material. When testing long components on the frame, additional dual bushings (4) are placed behind the cylinder block (2). The crossbeam of the hydraulic motor moves in the guides of the support (1), moved by hydraulic jack (6). The installed rotary cam engine (7) allows the rock bolt to rotate around its own axis. There is the possibility of three areas of maximum axial tensile force: 400 kN, 800 kN and 1,600 kN. Maximum piston displacement is 800 mm. The maximum rate of extension of the measuring disk is 0.03 m/s. The design of dual bushings enables the release of the upper and lower parts (Fig. 2). The dual bushings were filled with a concrete compressive strength $R_c = 80$ MPa (Fig. 2).
2.2 Instrumentation

Force measurement at the laboratory test facility was carried out using four strain gauge force sensors. The sensors were placed every 90 degrees on the measurement disk (Fig. 3). The total force recorded during the bolt rod tension tests is the sum of the forces generated at the various sensor strengths. Type FFM 50 sensors were used in the test. Each of the sensors mounted is characterized by a nominal load equal to 500 kN. Each sensor was calibrated on the AG Walter + Bai DB 3000/200 pull out machine by controlled compression. As part of the test, a load scheme, which consisted in determining the measure of increase in force at every 5 kN, was established. The crosshead speed was 0.5 kN/s, while the retention time was 10s. Control sensor calibration was carried out by comparing two independent indicators, the pull out machine sensor and the measuring amplifier. The sensors were tested to 50% of nominal value (250 kN).

The second test validating the correctness of the reading of the acting forces consisted of resistance tests on the tensile strength of the bolt rod and specially prepared samples made of the same material (Fig. 4). The middle section of each sample (at a length of 100 mm) was rolled to a diameter of 10 mm. The study involved comparing the forces required to break off the sample on the loading machine and the HUK stand.

Figure 1 Laboratory stand HUK for rock bolt testing (tensioning with force-blue arrow). 1 – frame, 2 – block of hydraulic jacks, 3 – piston, 4 – dual bushings, 5 – hydraulic motor crossbeam, 6 – hydraulic jacks, 7 – rotary engine, 8 – spherical plate, 9 – three jawed clamp, 10 – displacement (encoder), 11 – force sensor, 12 – rock-bolt with deformable plate.

Figure 2 Dual bushings—a rock bolt fixed in a concrete block (photo: K. Skrzypkowski)
The results obtained in the tests are in the range of 699 MPa to 703 MPa and overlap in terms of the tensile strength of ATLAS III steel grade (Rm = 697÷705) (Certificate of receipt, 2012).

Displacement measurements of support elements (in the absence of sufficient dilation of the expansion head in the block) and the tension of the bolt rod were carried out using a HLS-S-10-01 type incremental line encoder. The encoder, permanently attached to the cylinder block (2), can measure maximum displacement at 1000 mm/s speed, while its resolution is 0.1 mm/impulse (Fig. 5).

In order to determine the material deformation of the components tested (bolt rod), a lattice type electro-resistant strain gauge (Fig. 6) made of constantan wire (60% Cu + 40% Ni) with a diameter of 0.025 mm was used in the tests. The resistance wire was placed on a paper support pad (100% tissue paper). The tips supplying electric current were made of coil wire (tin-plated copper tape with a width of 0.55 mm and a thickness of 0.15 mm). Strain gauges were characterized by resistance of 120 Ω, measurement base length of 10 mm and sensitivity factor of deformation of k = 2.15. The choice of a lattice strain gauge resulted from the following technical characteristics. Lattice type strain gauges are not sensitive to lateral deformation; however, the hose-type strain gauges are sensitive to deformation perpendicular to their length. The rod surface was thoroughly cleaned with fine sandpaper so that the strain gauge could be attached to the bolt rod. After the surface was mechanically cleaned, it was subjected to chemical purification so that
degreasing with acetone could be carried out. The cleaned metal surface was coated with a thin layer of a special quick-drying adhesive (HBM Z70). Alongside axial tensioning of the bolt rod, the strain gauges were connected to the Quantum X 840 measuring amplifier in such a manner that the active t1 strain gage was attached alongside the axis of the stretching rod and the compensatory strain gauge t2 was attached to an idle piece of material identical to the tested one and placed near to the active strain gauge t1.

Figure 5  Line encoder installed on the HUK machine (photo: K. Skrzypkowski)

Figure 6  Temperature compensation—rock bolt with three measurement bases (photo: K. Skrzypkowski). I—first measuring base of strain gauges (I1), II—second measuring base of strain gauges (I2), III—third measuring base of strain gauges (I3)

During the process of rock bolt expansion, the results of force measurement of displacement and deformation were recorded continuously by a program specialized in the field of measurement technology—known as “CATMAN-EASY”. The choice of program resulted from the possibility of cooperating with MS Windows operating system and connecting the computer with the QuantumX MX840 universal measuring amplifier via an Ethernet line (Hottinger Baldwin Measurement, 2012). The program enabled on-line visualization and evaluation of the measurement. In addition, after the tests were completed, reports documenting the results of the measurements were created and then stored as an ASCII file. Following this, the data was transferred to Microsoft Excel for results analysis.

3. Laboratory test facility for rock bolt testing

Similarity of loading conditions of the single rock bolts installed in the underground excavation and in the laboratory stand has been fulfilled by arrangement of two parameters—strength of the block and tensional force applied for its loading. Rock bolts installed in the mine first transmit axial tensional forces, which has been assured in the laboratory tests. In the case of mechanically end-anchored roof bolts, the contact
strength of the head/rock mass at the position of the anchor is critical. The compressive strength of the concrete block, where the bolt was installed, satisfied in situ strength of the rock mass.

Static tests of expansion rock bolts were carried out at the laboratory test stand simulating mining conditions. The support consists of an RS-2N type rock bolt with a length (l) from 1,823 mm to 1,834 mm and a diameter (d) from 18.24 mm to 18.29 mm (Table 1).

**Table 1: Technical parameters of RS-2N rock bolts**

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Characteristics</th>
</tr>
</thead>
<tbody>
<tr>
<td>Steel grade</td>
<td>ATLAS III</td>
</tr>
<tr>
<td>Cross-section</td>
<td>Round, smooth</td>
</tr>
<tr>
<td>Rod diameter d (actual), mm</td>
<td>18.24±18.29</td>
</tr>
<tr>
<td>Rod length l (actual), mm</td>
<td>1823±1834</td>
</tr>
<tr>
<td>Thread*</td>
<td>M20, cold rolled</td>
</tr>
<tr>
<td>Thread length l, mm</td>
<td>170</td>
</tr>
<tr>
<td>Active rod length subject to extension l0, mm</td>
<td>1670</td>
</tr>
<tr>
<td>Core diameter of bolt rod d3 (actual), mm</td>
<td>16.45±16.53</td>
</tr>
<tr>
<td>Outer diameter of bolt rod d2 (actual), mm</td>
<td>19.70±19.86</td>
</tr>
<tr>
<td>Thread length (actual), mm</td>
<td>168±170</td>
</tr>
<tr>
<td>Tensile strength, Rm, MPa</td>
<td>770</td>
</tr>
<tr>
<td>Yield strength Re (R0.2), MPa</td>
<td>440±448</td>
</tr>
<tr>
<td>Elongation after rupture**, A5, %</td>
<td>21.2±21.4</td>
</tr>
<tr>
<td>Mass 1 mb, kg</td>
<td>2.043±2.050</td>
</tr>
</tbody>
</table>

Note:* A crumpled zone is formed as a result of cold rolling—thanks to which the material is hardened (plastic deformation causes an increase in the density of the crystallographic network of defects). An increase in the dislocation density causes metal strengthening since a reduction in the distance between dislocations takes place, and hence there is an increase of strength as a result of interaction between them (Wendorff, 1968; Przybyłowicz, 1992).

The bolt rod has an M20 cold rolled thread on one end, while the other is swollen (with square cross-section profile) and adapted for transmitting torque (Fig. 7). The rod cooperates with the KE3-2K type expansion head 136 mm long with a diameter of 36 mm (Table 2, Fig. 8) and is composed of three jaws, two K-2143-2-2 type expanders, 31 mm φ springs and a conical cap. Round profiled bolt washers with a thickness of 6 mm were used in the studies (Table 3, Fig. 9). The choice of this shape of washer with a rock bolt support resulted from its common use in the copper ore mines.

![Figure 7 RS-2N bolt rod](image-url)
### Table 2  Technical characteristics of the KE3-2K expansion head

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Characteristics</th>
</tr>
</thead>
<tbody>
<tr>
<td>Length of head, mm</td>
<td>136</td>
</tr>
<tr>
<td>Diameter of head, mm</td>
<td>36</td>
</tr>
</tbody>
</table>

**a) Mechanical parameters of KE3-2K GG81/95 rock bolt jaws**

<table>
<thead>
<tr>
<th>Type of material</th>
<th>White malleable cast iron</th>
</tr>
</thead>
<tbody>
<tr>
<td>Tensile strength $R_m$, MPa</td>
<td>521-534</td>
</tr>
<tr>
<td>Elongation, %</td>
<td>7.2-7.8</td>
</tr>
<tr>
<td>Hardness, HB</td>
<td>167-176</td>
</tr>
</tbody>
</table>

**b) Mechanical camshaft parameters K-2143-2-2a**

<table>
<thead>
<tr>
<th>Type of materials</th>
<th>White malleable cast iron</th>
</tr>
</thead>
<tbody>
<tr>
<td>EN-GJMW-400-5 wg PN-EN 1562:2000</td>
<td>White malleable cast iron</td>
</tr>
<tr>
<td>Working length of thread, mm</td>
<td>26.50</td>
</tr>
<tr>
<td>Tensile strength $R_m$, MPa</td>
<td>521-539</td>
</tr>
<tr>
<td>Elongation, %</td>
<td>7.2-7.9</td>
</tr>
<tr>
<td>Hardness, HB</td>
<td>163÷176</td>
</tr>
</tbody>
</table>

### Table 3  Technical characteristics of rock bolt washer

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Technical characteristics</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Round washer</td>
</tr>
<tr>
<td>Steel grade</td>
<td>S235JR</td>
</tr>
<tr>
<td>Washer thickness $t$, mm</td>
<td>6.00</td>
</tr>
<tr>
<td>Free height of uncompressed washer $H$, mm</td>
<td>16.60</td>
</tr>
<tr>
<td>Inner diameter $d$, mm</td>
<td>22.00</td>
</tr>
<tr>
<td>Outer diameter $D$, mm</td>
<td>141.00</td>
</tr>
<tr>
<td>Side length $a$, mm</td>
<td>–</td>
</tr>
<tr>
<td>Collaborative diameter of disk spring $d_l$, mm</td>
<td>93.30</td>
</tr>
<tr>
<td>Tensile strength, $R_m$, MPa</td>
<td>403÷442</td>
</tr>
<tr>
<td>Yield strength $R_e$, MPa</td>
<td>314÷338</td>
</tr>
<tr>
<td>Elongation of samples after rupture $A_5$, %</td>
<td>36.2÷39.0</td>
</tr>
</tbody>
</table>
In order for laboratory studies to best satisfy mining conditions, dual bushings (Fig. 2) filled with concrete having a compressive strength of at least 80 MPa were used at the laboratory test facility. Borehole with a diameter of 37 mm to 38 mm were made in the 220 mm diameter cylindrical concrete blocks. The axially arranged boreholes had the same diameter as the real ones in mining rooms. PVC tubes were used to protect the boreholes while setting concrete. The tubes were removed after a period of “setting” of the concrete mixture and before tests. The procedure for installing the rock bolt support commenced after a period of 28 days from the time of filling the dual bushings. The instrumented rock bolts were fed manually into the concrete block by a measuring disk and an opening in the hydraulic rock bolt load support arrangement (HUK). Afterwards, the rock bolts were fixed in the borehole with a torque of 250Nm, in accordance with the requirements for expansion rock bolts used in metal ore mines in Poland. The differences between the diameters of the rock bolt borehole and the diameter of the head did not exceed 2 mm. A difference greater than 2 mm caused broaching of the rock bolt by jaw head expansion (chamfering of threads). This is due to the structure of the three jaw heads which block the axial movement of the rod. The head installed in the concrete block is expanded, whereas the other end of the rock bolt, preceded by a profiled washer, is placed on a retractable disk of the HUK stand. After expansion of the head in the borehole, initial tension was applied with force 30 kN. Next, the value of the 400 kN static tension force was set on control panel I. Static tensile tests of the rock bolt support consisted of intermittent increases in pressure of 10 bar. Each increase in pressure was preceded by a 10 second break. The duration of the test up to the breaking point was an average of 120 seconds and was within the range of both the requirements imposed by the PN-EN 10002-1 (2004) standard for tensile tests and the corresponding growth of static load in the excavation room. During the entire load cycle, the expansion force value was continually measured using four force KKM 50 type sensors mounted on the retractable disk of the hydraulic unit (HUK). Furthermore, the distortion value of the rod was recorded with the help of three electro-resistant strain gauges attached to its rolled part at equal distances of 400 mm between each other. In addition, the station was equipped with a one-line encoder, recording the total elongation to rupture. For the entire duration of the static stretching process, all the data from the various sensors was recorded using “CATMAN-EASY”, a specialized software in the field of measurement technology.

The following definitions were introduced to facilitate analysis of the results:

- **Bolt rod diameter, d**—the diameter of the rolled rod
- **Bolt rod diameter, D3**—the smallest diameter of the rod; in the case of a rod with an M20 thread, it is the thread root diameter
- **Active length of bolt rod, l0**—the length subjected to tensile stress, the section of dilation of the concrete block head to the rod flange from the side of the washers (Fig. 7),
- Breaking force, $F$—the border force which was followed by breaking the continuity of the material (load limit value)
- Displacement, $\Delta l$—the value of the maximum breaking elongation of the rock bolt

The rock bolt fixed in a concrete block, which has one degree of freedom, is subjected to axial tensile force. Dilation is so sufficient that during the study there was no slide of the rock bolt from the borehole and a rupture of the thread followed (Fig. 10).

![Figure 9 Profiled rock bolt washer](image)

**Figure 9** Profiled rock bolt washer

Load speed, $v$—the ratio of the displacement $\Delta l$ or $\Delta l_u$ recorded during the test period:

$$v = \frac{\Delta l}{t}, m/s$$

(1)

Maximum tensile stress, $\sigma_r$—the ratio of maximum breaking force $F$, impacting the smallest cross sectional area of the bolt rod (Osiński et al., 1975):

$$\sigma_r = \frac{F}{\pi \cdot d^2}, MPa$$

(2)

Relative strain of rock bolt, $\varepsilon$—the ratio value of total displacement (breaking elongation) of bolt rod $\Delta l$ in relation to the active length of the rod $l_0$. Relative strain is the sum of the deformation of the rolled part and the thread of the rod:

$$\varepsilon = \frac{\Delta l}{l_0} \cdot 100\%$$

(3)

Relative strain of the bolt rod, $\varepsilon_1, \varepsilon_2, \varepsilon_3$ (%)—the value of bolt rod deformation in the scope of resilience and linearity interpreted from an electro-resistant strain gauge attached to the rolled surface (d) of the rod.

Examples of the characteristic parameters obtained in the tests are shown in Table 4 and in Figure 11. The variation scope of breaking force $F$ ranged from 160.3 to 164.5 kN. Relative strain $\varepsilon$ ranged from 5.7% to 6.7%. Static load speed was in the range of 0.00089 m/s to 0.00094 m/s.
Table 4  Technical characteristics of the rock bolt support expanded statically

<table>
<thead>
<tr>
<th>Lin</th>
<th>Parameter</th>
<th>Unit</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.</td>
<td>Bolt rod length, ( l )</td>
<td>mm</td>
<td>1825</td>
</tr>
<tr>
<td>2.</td>
<td>Active bolt rod length, ( l_0 )</td>
<td>mm</td>
<td>1670</td>
</tr>
<tr>
<td>3.</td>
<td>Bolt rod diameter, ( d )</td>
<td>mm</td>
<td>18.23</td>
</tr>
<tr>
<td>4.</td>
<td>Bolt rod diameter, ( d_3 )</td>
<td>mm</td>
<td>16.43</td>
</tr>
<tr>
<td>5.</td>
<td>Breaking force, ( F )</td>
<td>kN</td>
<td>160.382</td>
</tr>
<tr>
<td>6.</td>
<td>Displacement, ( \Delta l )</td>
<td>mm</td>
<td>109.728</td>
</tr>
<tr>
<td>7.</td>
<td>Tensile stress time to moment of rock bolt rupture, ( t )</td>
<td>s</td>
<td>117.14</td>
</tr>
<tr>
<td>8.</td>
<td>Load speed, ( \nu )</td>
<td>m/s</td>
<td>0.00094</td>
</tr>
<tr>
<td>9.</td>
<td>Maximum tensile stress, ( \sigma_r )</td>
<td>MPa</td>
<td>756.468</td>
</tr>
<tr>
<td>10.</td>
<td>Relative deformation, ( \varepsilon )</td>
<td>%</td>
<td>6.571</td>
</tr>
<tr>
<td>13.</td>
<td>Force corresponding to the deformation of the electro-resistant strain gauge ( \varepsilon_1, \varepsilon_2, \varepsilon_3 ), at the level of 2‰, ( F_s )</td>
<td>kN</td>
<td>98.040</td>
</tr>
<tr>
<td>14.</td>
<td>Displacement corresponding to electro-resistant strain gauge ( \varepsilon_1, \varepsilon_2, \varepsilon_3 ), at 2 ‰, level ( \Delta l )</td>
<td>Mm</td>
<td>15.820</td>
</tr>
<tr>
<td>15.</td>
<td>Tensile stress time in the scope of elasticity, ( t_s )</td>
<td>s</td>
<td>55.80</td>
</tr>
</tbody>
</table>

Figure 10  View of ruptured rod surface (photo: K. Skrzypkowski)
From the analysis of the course of the characteristics shown in Fig. 11, it can be concluded that they differ from each other in both the current values and the inclination of the curves. Strains: $\varepsilon_1$, $\varepsilon_2$ and $\varepsilon_3$ are based on measurements made using the electro-resistant strain gauges attached directly to the surface of the steel rod (properly prepared beforehand) and a relatively short measurement database, which in this case was 10 mm, placed centrally along the entire length of the rod at distances of every 400 mm. Deformations of individual strain gauges are therefore accepted as equivalent of the deformation of the steel rod (bolt rod). From the graph it appears that all three courses practically coincide, with uniformly increasing deformation to a value of less than 0.2% of the base length and a load range to the tension value of about 370 MPa, which responds to the axial force value of 100 kN, influences the rock bolt.

A real total displacement/elongation of the rock bolt is much bigger than bolt elongation calculated directly from the steel rebar strains, measured with the attached tensometer. Such a situation is similar to mining excavation and layers of rock coupled with bolts within the reinforced zone (artificial beam). The encoder records ($\varepsilon_{RB}$) correspond to the sum of displacements due to the deformation of washer, deformation of threaded part of the rod with a smaller diameter and the active cross-section, as well as a result of imperfect contact of the head with the walls of the borehole.

4. Conclusion

The stress-strain characteristics of a rock bolt, based on measurements carried out with a precise line encoder and a resistant tensometer, have dramatically different courses. In the first case of the measurement, the result is a sum of the elongation of the rod itself, the elongation of the threaded part of the rod with a smaller diameter, the deformation of the washer, and the displacement of the head due to its imperfect contact with surrounding rock walls in the borehole. In this case we are talking about the characteristics of the rock bolt and not the bolt rod. In the strain-stress course, one can observe two distinctive parts. The first is in the range of up to about 400 MPa stress and a strain of up to about 1%. The second is at a higher stress and larger deformation of up to about 6.5% (not analysed in this paper). Rock bolt loads greater than 100 kN cause increasingly rapid deformation of the structural elements of the rock bolt, in particular the bolt washers, but also the threaded part together with the nut. Certain parts of the displacement recorded by the encoder also result from a slight displacement of the rock bolt head.

The modern laboratory test stand for rock bolt examination enables:
• rock bolts testing in real scale and length up to 6m
• studying of rebar bolts, cable or string bolts, frictional and made of various materials and constructions
• loading with static and dynamic force
• utilizing three loading ranges of up to 400 kN, 800 kN and 1,600 kN axial tension force

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The contact problem of the supported circular cylindrical underground opening in elastic rock

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M.S. Stavropoulou, University of Athens, Hellas

Abstract

The mixed boundary value problem revisited here pertains to the interaction of an elastic plain or composite closed ring with a circular cylindrical opening in an elastic rock mass. The elastic ring is assumed to behave like a thin shell that is able to carry out both thrust and bending moment. The contact between the ring and the rock is either perfect or adhesive (i.e. no-slip occurs along the contact of the two elastic bodies), or non-perfect (i.e. full-slip is permitted between the ring and the medium). Also, the Kirchhoff–Love assumptions governing the mechanical behaviour of elastic thin shells are considered here to hold true. For both limiting contact conditions the equilibrium for the thin shell takes the form of a pair of differential equations, while the stress equilibrium equation for the elastic ground is expressed by a single complex integral equation. It is explicitly shown that the difference among the tangential stress acting on the ring for the full-slip contact conditions and for the no-slip, lies in a term containing the derivative of the tangential displacement jump along the circular interface. If the latter jump is null, then the equations for the frictionless-slip degenerate to those of the no-slip contact. The solution of both contact problems is achieved by virtue of a special complex boundary collocation method. Two example cases are illustrated referring to the plain supporting ring and the composite liner such as shotcrete reinforced with steel sets. The delay of installation of the closed ring is also taken into account. The merit of the proposed collocation method is that it may be easily programmed and hence used for the selection of appropriate tunnel liner properties. Various design charts may be produced in this manner to better facilitate the design of thin liners in everyday tunnelling design practice or benchmarking numerical codes employed for this purpose for the simple cases of elastic continuous ground conditions. However, it is demonstrated that the proposed solution could be generalized in a straightforward manner to consider the presence of cracks or joints exhibiting stress singularities at their crack tips, in the surrounding rock mass close to the tunnel’s periphery. This generalization cannot be easily achieved by currently available computational algorithms.

1 Introduction

The design of tunnel or borehole lining is a problem of major technological importance in mining, tunnelling and reservoir engineering. Historically speaking, the problem of strengthening of a circular hole in an infinite elastic isotropic plane by a perfectly bonded elastic ring was solved in closed-form by Savin (1961) by virtue of complex potential functions; the full-slip boundary value problem was not considered by Savin in his textbook. However, both these extreme contact conditions are expected to bracket the expected stresses that occur in real practice with the value of the interfacial friction angle ranging between zero (full interfacial slip) and infinity (no interfacial slip).

Plane strain analyses based on the elastic continuum model have been reported for tunnels supported by a continuous liner (e.g., Hoeg 1968; Einstein and Schwartz 1979) and these have been used in the design of tunnel liners and buried pipes. Furthermore, the response of circumferential ribs used to support a long circular tunnel in elastic ground was investigated by Moore (1994). A rib-ground interaction solution was described there for biaxial field stresses. A simplified procedure was also developed for predicting thrust and moment in rib supports. Einstein and Schwartz (1979) have solved both the two extreme mixed-boundary value problems of full-slip and no-slip conditions along the support-ground interface while they
have considered the support as a ‘thick shell’. For this purpose they have employed Michell’s generalized stress function (Timoshenko and Goodier 1951) and they have imposed ad-hoc symmetry and periodicity conditions of the contact problems. Further, a methodology was recently proposed for the mechanical analysis of composite supports, such as liners consisting of sprayed concrete or shotcrete and steel sets by Carranza-Torres and Dieterichs (2009). It is based on the ‘equivalent section’ approach based essentially on Voigt’s summation formula that is an established technique of structural analysis. This methodology perfectly suits the presented solution since it may facilitate the calculation of the equivalent elastic parameters of the composite ring that are subsequently used as input for the solution of the ring-rock contact problem.

In this work we study the continuous or cracked elastic rock with a lined tunnel analytically up-to a significant level, based on the complex variable theory developed by Muskhelishvili (1963). The support lining is assumed here to behave like a thin shell. The behaviour of the thin shell is depicted by the Kirchhoff–Love hypotheses for thin shells. It is remarked here that there are two different classes of shells i.e. thick shells and thin shells. A shell is called thin if the maximum value of the ration of shell thickness over the hole radius (i.e. \(h/R\) with the notations shown in Figure 1) can be neglected in comparison with unity. Correspondingly, shells will be called thick whenever such terms cannot be neglected. According to Novozhilov (1964) for ordinary technical calculations it is admissible to have an error of 5% and for thin shells this corresponds to maximum relative shell thickness of \(h/R \leq 1/20\).

![Figure 1](image1.png)

**Figure 1**  Schematic diagram of a reinforced circular cylindrical underground opening in an infinite elastic body transected by a number of cracks or joints (\(\ell_i, i = 1,...,n\)) and subjected to far-field principal stresses and Cartesian, as well polar coordinate systems.

The contact between the ring and the rock is either perfect or adhesive (i.e. no-slip occurs along the contact of the two elastic bodies) or non-perfect (i.e. full-slip is permitted between the ring and the medium). It is remarked that both these solutions of stresses are expected to bracket the expected stresses which occur in practice with the value of the interfacial friction angle ranging between zero (full interfacial slip) and infinity (no interfacial slip). It is demonstrated that for both limiting contact conditions, the equilibrium for the thin shell takes the form of a pair of differential equations, while the stress equilibrium equation for the elastic ground is expressed by a single complex integral equation. It is also explicitly shown that the difference among the tangential stress acting on the ring for the full-slip contact conditions and for the no-slip, lies in a term containing the derivative of the tangential displacement jump along the circular interface.
If the latter jump is null, then the equations for the frictionless-slip degenerate to those of the no-slip contact. The solution of both contact problems is achieved by virtue of a special complex boundary collocation method. Two example cases are subsequently illustrated referring to the plain supporting ring and the composite liner such as shotcrete reinforced with steel sets. The delay of installation of the closed ring is also taken into account. The merit of the proposed collocation method is that it may be easily programmed and hence used for the selection of appropriate tunnel liner properties. Various design charts may be produced in this manner to better facilitate the design of thin liners in everyday tunnelling design practice or benchmarking numerical codes employed for this purpose for the simple cases of elastic continuous ground conditions.

It is demonstrated that the proposed solution could be generalized in a straightforward manner to consider the presence of cracks or joints exhibiting stress singularities at their crack tips, in the surrounding rock mass close to the tunnel’s periphery. This generalization cannot be easily achieved by currently available computational algorithms.

2 The no-slip and full-slip contact of a thin shell with an opening

2.1 Formulation of the problem

This problem pertains to the analysis of a lined circular underground opening in a linear elastic isotropic ground in plane strain conditions. In order to formulate the problem we consider a circular cylindrical opening of radius $R$ that is supported with a closed circular thin shell of thickness $h$ and subjected to primary principal stresses $\sigma_1, \sigma_2$ at infinity (Figure 1). The assumption of constant far-field stresses is justified for deep tunnels (i.e. at large depth compared to the tunnel’s radius). Both the components of this system (i.e. ground-support) are assumed to behave as isotropic, linear elastic solids with Young’s moduli, Poisson’s ratios and shear moduli denoted by the symbols, $E, v, G$ and $E_1, v_1, G_1$, for the ground and the thin shell support, respectively. The reinforced body is referred to a Cartesian coordinate system $Ox_1x_2$ and a polar coordinate system $Or\theta$ as is shown in Figure 1. Also, the part of the plane which lies to the left of the oriented (anti-clockwise) hole boundary $\gamma$ is denoted by $S^+$, and that located to the right by $S^-$. Accordingly, the kinematic and static quantities have a superscript (+) or (-) depending on the side of the contour of the hole they refer to. The rock may contain $n$ curvilinear joints or cracks $\ell_j$ $(j = 1, ..., n)$ of arbitrary shape and location, where $k$ cracks $\ell_1, ..., \ell_k$ may lie at close distances along a path simulating a single crack with intact rock bridges between fractured open segments of the joint. The locations and extent of the most critical cracks and joints are assumed to be known from a previous mapping of disc cores and/or geophysical logs (like georadar, seismic or other); as is expected, in the lining design the most critical cracks are those that should be considered while the other could be accounted by suitably adjusting the elastic and strength parameters of the ground in a usual upscaling procedure (e.g. Mori-Tanaka type or similar).

The following boundary and far-field conditions of the problem should be considered:

(i) The inner surface of the elastic ring is tractionless, i.e. the following boundary condition in complex form is valid

\[
\sigma_n^+ - i\sigma_\ell^+ = 0
\]  

(1)

where $i = \sqrt{-1}$ is the usual imaginary unit, and $Ont$ is a local orthogonal coordinate system moving along the boundary of the hole (e.g. Figure 1).
The contact problem of the supported circular cylindrical underground opening in elastic rock

G.E. Exadaktylos and M.S. Stavropoulou

Ground Support 2016, Luleå, Sweden

(iiia) For perfect no-slip or perfect adherence contact conditions at the ring-body interface the tangential (hoop) strains of the contacting bodies as well as the radial stresses must be identical, that is

\[ \varepsilon_\theta^+ = \varepsilon_\theta^- = \varepsilon_\theta^{(str)}, \quad \sigma_n^+ = \sigma_n^- \] (2)

By applying Hooke’s law for the case of plane strain it is obtained (Timoshenko and Goodier 1951)

\[ \varepsilon_\theta^{(str)} = \varepsilon_\theta^+ = \varepsilon_\theta^- = \frac{1}{E} \left[ (1 - \nu^2) \sigma_\theta - \nu \left( 1 + \nu \right) \sigma_r \right] \] (3)

wherein \( \sigma_\theta = \sigma_n, \sigma_r = \sigma_s \) denote the circumferential (hoop) stress and the radial stress components, respectively, acting at the elastic body’s side (e.g. Figure 1) and the rest of the symbols have been defined previously.

(iiib) For full-slip contact conditions at the ring-body interface the radial displacements, \( u_r^+ \), and stresses are equal in both bodies, whereas the shear stress vanishes, i.e.

\[ u_r^+ = u_r^- , \quad \sigma_n^+ = \sigma_n^- , \quad \sigma_r^+ = \sigma_r^- = 0 \] (4)

(iii) The ground is subjected to the far-field principal stresses \( \sigma_1, \sigma_2 \) as is illustrated in Figure 1.

(iv) The normal and shear tractions applied along the crack boundaries \( \ell_j (j = 1, \ldots, n) \) are known, i.e.

\[ \sigma_n^+ - i \sigma_r^+ \bigg|_{\ell_k} , \quad k = 1, \ldots, n \] (5)

Of outmost importance here are cracks with possible fluid pressure that could produce hydraulic fracturing of the rock mass behind the liner. The constitutive relations of a thin shell in the frame of the Kirchhoff–Love-Gol’denveizer-Novozhilov theory take the following simple form

\[ T_1 = T(\theta) = E_s \varepsilon_1 = E_s \varepsilon_\theta^{(str)}, \quad T_2 = E_s \nu \varepsilon_1 = E_s \nu \varepsilon_\theta^{(str)}, \]

\[ M_1 = M(\theta) = D_s \kappa_1, \quad M_2 = D_s \nu \kappa_1, \]

\[ E_s = \frac{E_i h}{1 - \nu_i^2}, \quad D_s = \frac{E_i h^3}{12(1 - \nu_i^2)} \] (6)

in which \( T \) denotes the specific axial thrust and \( M \) the bending moment, whereas small terms of the order of \((h/R)^2\) have been discarded for the considered case of thin shells, \( \varepsilon_\theta^{(str)}, \kappa_1 \) denote the tangential (hoop) strain and curvature of the middle surface of the cylindrical support ring, the elastic stiffness of the shell is denoted by \( E_s \) and the flexural rigidity of the shell denoted by the symbol \( D_s \). Finally, after
considering the traction boundary conditions given by Eq. (1), the governing differential equations for the specific axial thrust $T$ and bending moment $M$ along the contact zone $\gamma$ of the two bodies have been derived as follows

$$\begin{align*}
\frac{1}{R} \frac{\partial T(\theta)}{\partial \theta} - \frac{1}{R^2} \frac{\partial M(\theta)}{\partial \theta} - \sigma_{\theta}^- &= 0, \\
\frac{T(\theta)}{R} + \frac{1}{R^2} \frac{\partial^2 M(\theta)}{\partial \theta^2} - \sigma_n^- &= 0
\end{align*}$$

(7)

2.2 Equations for the ring: No-slip case

The tangential stress acting along the interface may be found from the Eq. (3) giving the hoop strain along the interface, and from the continuity of normal stresses in the following manner

$$\sigma_\theta^+(h/2) = \frac{E_i}{E} \left( \frac{1-v_t^2}{1-v_i^2} \right) \sigma_\theta^- + \left[ \frac{v_t (1+v_t)}{E} - \frac{v_t (1+v_i)}{E} \right] \sigma_r^-$$

(8)

where $\sigma_\theta^+$ is assumed to act at the mid-height of the support-section and we also assume that it coincides (the mid-height) with the wall of the tunnel. Substituting into the system of Eq’s (15) the expression of the interfacial hoop stress as is given by the above relation (8), and the expression for the thrust force as is given by the constitutive relations (6) and (3), as well as the strain-stress relationship (3), we finally obtain the final system of governing partial differential equations for the ring under no-slip interface conditions

$$\begin{align*}
&-\frac{h}{R} \left[ \frac{E_i}{E} \frac{v_t (1+v_t)}{1-v_i^2} + \frac{h}{R} \frac{v_t (1+v_t)}{1-v_i^2} \right] \frac{\partial \sigma_n^- (\theta)}{\partial \theta} + \\
&\frac{E_i}{E} \frac{h}{R} \frac{1-v_t^2}{1-v_i^2} \frac{\partial \sigma_\theta^- (\theta)}{\partial \theta} - \sigma_r^-(\theta) = 0, \\
&\left[ \frac{1}{E} \frac{E_i}{R} \frac{v_t (1+v_t)}{1-v_i^2} \right] \sigma_n^- (\theta) - \frac{1}{6} \left( \frac{h}{R} \right) \frac{v_t (1+v_t)}{1-v_i^2} \frac{\partial^2 \sigma_n^- (\theta)}{\partial \theta^2} + \\
&\frac{E_i}{E} \frac{h}{R} \frac{1-v_t^2}{1-v_i^2} \sigma_\theta^- (\theta) = 0
\end{align*}$$

(9)

(10)

in which $\sigma_r^- = \sigma_r$, $\sigma_\theta^-$, $\sigma_{\theta}^- = \tau_{r\theta}$ denote the radial, tangential (hoop) and shear stresses acting on the elastic ground. Finally, the tangential stress acting on the shell at its middle surface may be obtained in the following fashion

$$\sigma_\theta^+(0) \approx \frac{T}{h} = \frac{1}{1-v_i^2} \left( \frac{E_i}{E} \left[ (1-v_t^2) \sigma_\theta^- - v_t (1+v_t) \sigma_r^- \right] \right)$$

(11)
2.3 Equations for the ring: Full-slip case

Based on the following tangential strain expression in terms of displacements in polar coordinates (Timoshenko and Goodier 1951), and the first of interfacial conditions (4), we derive the relationship giving the hoop strain acting along the hole boundary in the following manner

\[
\varepsilon_{\phi}^{(sw)} = \varepsilon_{\phi}^+ = \frac{u^+}{R} + \frac{1}{R} \frac{\partial u^+_\theta}{\partial \theta} = \frac{u^-}{R} + \frac{1}{R} \frac{\partial u^-_\theta}{\partial \theta} = \varepsilon_{\phi}^- + \frac{1}{R} \frac{\partial}{\partial \theta} (u^+_\theta - u^-_\theta)
\]  

(12)

In addition the tangential stress acting along the interface may be found from the relationship above, if the following relationships are taken into account

\[
\varepsilon_{\phi}^{(sw)} = \varepsilon_{\phi}^+ = \frac{1}{E_1} \left[ \left(1 - v_i^2\right) \sigma_{\phi}^+ - v_i \left(1 + v_i\right) \sigma_r^+ \right]
\]

\[
\varepsilon_{\phi}^- = \frac{1}{E} \left[ \left(1 - v^2\right) \sigma_{\phi}^- - v \left(1 + v\right) \sigma_r^- \right]
\]  

(13)

and the second interfacial condition (4), in the following fashion

\[
\sigma_r^+(h/2) = \frac{E_1}{E} \left(\frac{1 - v^2}{1 - v_i^2}\right) \sigma_r^+ + \left[ \frac{v_i \left(1 + v_i\right)}{\left(1 - v_i^2\right)} - \frac{E_1}{E} \frac{v \left(1 + v\right)}{\left(1 - v^2\right)} \right] \sigma_r^- + \\
+ \frac{E_1}{\left(1 - v_i^2\right) R} \frac{\partial}{\partial \theta} (u^+_\theta - u^-_\theta)
\]  

(14)

The final differential eqns for the ring in this case take the final form

\[
-\left[ \frac{E_1}{E} \frac{v \left(1 + v\right)}{\left(1 - v_i^2\right)} + \frac{1}{R} \frac{h}{6} \frac{v_i \left(1 + v_i\right)}{\left(1 - v_i^2\right)} \right] \frac{\partial }{\partial \theta} \sigma_r^+(\theta) + \\
\frac{E_1}{E} \left(\frac{1 - v^2}{1 - v_i^2}\right) \frac{\partial}{\partial \theta} \sigma_r^+(\theta) + \frac{E_1}{R} \frac{\partial^2}{\partial \theta^2} (u^+_\theta - u^-_\theta) = 0,
\]  

(15)

\[
\left[ 1 + \frac{E_1}{E} \frac{h}{R} \frac{v \left(1 + v\right)}{\left(1 - v_i^2\right)} \right] \sigma_r^+(\theta) - \frac{1}{6} \frac{h}{R} \frac{v_i \left(1 + v_i\right)}{\left(1 - v_i^2\right)} \frac{\partial^2}{\partial \theta^2} \sigma_r^+(\theta) + \\
\frac{E_1}{E} \frac{h}{R} \frac{1 - v^2}{1 - v_i^2} \sigma_r^+(\theta) + \frac{h}{R} \frac{E_1}{\left(1 - v_i^2\right) R} \frac{\partial}{\partial \theta} (u^+_\theta - u^-_\theta) = 0
\]  

(16)

Also in this case the tangential stress acting at the middle surface of the shell may be obtained in the following manner
\[
\sigma^+(0) \approx \frac{T}{h} = \frac{1}{1-\nu^2} \frac{E_1}{E} \left[ (1-\nu^2) \sigma_x^+ - \nu (1+\nu) \sigma_y^+ \right] + \frac{E_1}{(1-\nu^2)} \frac{1}{R} \frac{\partial}{\partial \theta} \left( u_\theta^+ - u_\theta^- \right)
\]

Comparing Eq’s (8) and (11) referring to the no-slip case, with the corresponding Eq’s (14) and (17) for the full-slip or frictionless slip case, it may be observed that their difference lies in the last term of the latter equations containing the derivative of the tangential displacement jump along the circular interface. If the latter jump is null, then the equations for the frictionless-slip degenerate to those of the no-slip or infinite interfacial friction angle.

### 2.4 Equations for the jointed rock

In the context of the Complex Variable theory the complex stress functions \( \Phi_0(z) \), \( \Psi_0(z) \) wherein \( z \) denotes the complex position vector as is illustrated in Figure 1, are usually employed for the solution of boundary value problems of the Theory of Elasticity (Muskhelishvili 1963). These functions are defined in the following fashion

\[
\Phi_0(z) = \Phi(z) + \Gamma, \quad \Psi_0(z) = \Psi(z) + \Gamma',
\]

where \( \Phi(z) \big|_{z \to \infty} \approx o(1/z), \quad \Psi(z) \big|_{z \to \infty} \approx o(1/z), \) and

\[
\Gamma = \frac{1}{4} (\sigma_1 + \sigma_2), \quad \Gamma' = -\frac{1}{2} (\sigma_1 - \sigma_2)
\]

It is noted that this form of the complex potentials \( \Phi_0(z), \Psi_0(z) \) satisfy the far-field conditions (iii) postulated in Section (2.1), namely that the ground is subjected to pre-existing stresses \( \sigma_1, \sigma_2 \) as is illustrated in Figure 1. The function \( \Phi(z) \) can be defined in the form of Cauchy integral in order to satisfy the traction boundary conditions described by Eq. (1), that is to say

\[
\Phi^-(z) = -\frac{1}{2\pi i} \oint_\gamma \Phi^-(\tau) \frac{d\tau}{\tau - z} + \frac{1}{2\pi} \sum_{j=1}^n \int_{\gamma_j} g_j(\tau) \frac{d\tau}{\tau - z}
\]

wherein the unknown densities \( \Phi^-, g_j \ (j = 1, \ldots, n) \) that are in general complex quantities, refer to the hole and the cracks, respectively (e.g. Figure 1); the usual notation is followed here, namely the boundary values of \( \Phi_0(z) \) as \( z \) approaches the hole boundary are denoted as \( \Phi_0(\tau) \) (see Figure 1). It may be shown that the values of the densities \( g_j \ (j = 1, \ldots, n) \) at the crack tips give the mode-I and mode-II stress intensity factors. The expression for the stress complex potential along the hole contour \( \gamma \) at the side of the rock that satisfies the displacements and tractions equations, as well as the displacement and stress continuity equations is given by the expression
The contact problem of the supported circular cylindrical underground opening in elastic rock

G.E. Exadaktylos and M.S. Stavropoulou

\[
\Phi^- (t) = \frac{1 - \nu}{1 + \kappa} \left[ \sigma_n^- + \sigma_\theta^- \right] - \frac{i \sigma_\tau^-}{1 + \kappa} G \frac{\partial u_0}{\partial r} = f_1 (t) + f_2 (t) - if_3 (t) + if_4 (t), \quad t \in \gamma
\]

Wherein

\[
f_1 (t) = \frac{1 - \nu}{1 + \kappa} \sigma_n^- (t) = \frac{\sigma_n^- (t)}{4}, \quad f_2 (t) = \frac{1 - \nu}{1 + \kappa} \sigma_\theta^- (t) = \frac{\sigma_\theta^- (t)}{4},
\]

\[
f_3 (t) = \frac{1}{1 + \kappa} \sigma_\tau^- (t) = \frac{1}{1 + \kappa} \tau_{\tau \theta} (t), \quad f_4 (t) = \frac{2G}{1 + \kappa} \frac{\partial u_\theta (t)}{\partial r}
\]

in which \( \sigma_n^- = \sigma_n \), \( \sigma_\tau^- = \sigma_\theta \), \( \sigma_\theta^- = \tau_{\tau \theta} \) denote the radial, tangential and shear stresses acting on the elastic body at the circular contour. It may be noted that for the no-slip case the latter stress is null.

The final complex integro-differential eqn that should be satisfied along the contact contour \( \gamma \) on the side of the elastic body in the general case of the fractured ground has as follows

\[
\frac{1}{\pi i} \int_{\gamma} \left( f_1 (\tau) + f_2 (\tau) - if_3 (\tau) + if_4 (\tau) \right) d\tau - \frac{1}{\pi i} \int_{\gamma} \left( f_1 (\bar{\tau}) + f_2 (\bar{\tau}) + if_3 (\bar{\tau}) - if_4 (\bar{\tau}) \right) d\bar{\tau} + \frac{1}{\pi i} \int_{\gamma} \left( f_1 (\tau) + f_2 (\tau) + i f_3 (\tau) - i f_4 (\tau) \right) d\tau - \frac{1}{\pi i} \int_{\gamma} \left( f_1 (\bar{\tau}) + f_2 (\bar{\tau}) + i f_3 (\bar{\tau}) - i f_4 (\bar{\tau}) \right) d\bar{\tau} = 2 \left[ \left( \Gamma + \tilde{\Gamma} \right) + \frac{dt}{dt} \Gamma' \right], \quad t \in \gamma
\]

The remaining equations of the boundary value problem refer to the satisfaction of the traction boundary conditions given by Eq. (5) along the crack contours.

2.5 The solution of the tunnel lining in intact rock

At this stage we discard the possible cracks and consider the intact rock in order to simplify the overall presentation. The system of differential and integral eqns for the two boundary value problems along the
circular contour of the hole is solved by a special boundary collocation method. For this purpose we consider the following interpolation polynomial \( u_n(\varphi, t) \) of a sufficiently smooth complex-valued function \( \varphi(t) \) (i.e. in our case the real functions \( f_1, f_2, f_3, f_4 \)) around the contour \( \gamma \) with \( t = e^{i\varphi} \) (Ivanov 1976)

\[
\begin{align*}
u_n(\varphi, t) &= \sum_{k=-n}^{n} \alpha_k t^k, & \alpha_k &= \frac{1}{2n+1} \sum_{j=-n}^{n} \varphi(\tau_j) \tau_j^{-k}, \\
\tau_j &= \exp \left(2\pi j/(2n+1)\right), & t &= e^{i\varphi}
\end{align*}
\]  

(24)

The system of two differential Eq’s (9), (10) for the case of no-slip and Eq’s (15), (16) for the case of full-slip coupled with the complex integro-differential Eq. (23) for the case of intact rock may be easily transformed with the aid of the interpolation scheme (24) into a system of four linear algebraic equations with four unknowns along the interface boundary, namely \( f_i(t), i = 1,...,4 \) for each of the two cases. The solution of this system of \( (8n+4) \) eqns and \( (8n+4) \) unknowns in turn gives the stresses and the strain term along the interface boundary according to relations (22).

The distributions of radial, tangential and shear stresses in polar coordinates in the elastic medium, may be found by recourse to the following relationships (Muskhelishvili 1963)

\[
\begin{align*}
\sigma_r + \sigma_\theta &= 4 \text{Re} \Phi_0^-(z), \\
\sigma_r - i\tau_{r\theta} &= 2 \text{Re} \Phi_0^+(z) - e^{2i\varphi}[\text{Re} \Phi_0^+(z) + \Psi_0^+(z)]
\end{align*}
\]  

(25)

Based on the results obtained previously, that is the expressions (21) and (28) for \( \Phi_0^-(z) \), the relation for \( \Phi_0^+(z) \) in terms \( \Phi_0^-(z) \) (Ioakimidis 1976), as well as the boundary collocation method expressed by relations (24), it is easy to find the expressions for the complex potentials \( \Phi^-(z) \) and \( \Psi^-(z) + \Psi^+(z) \) as follows

\[
\Phi^-(z) = \frac{1}{2\pi i} \int_{\gamma} \frac{\Phi^+(\tau) \, d\tau}{\tau - z} = \frac{1}{2\pi i} \int_{\gamma} \left( f_1(\tau) + f_2(\tau) - if_3(\tau) + if_4(\tau) \right) \frac{d\tau}{\tau - z}
\]

\[
= \frac{1}{2n+1} \sum_{k=-n}^{n} \sum_{j=0}^{2n+1} (f_1(\tau_j) + f_2(\tau_j) - if_3(\tau_j) + if_4(\tau_j))
\]

\[
= \frac{1}{2n+1} \sum_{j=0}^{2n+1} \left( f_1(\tau_j) + f_2(\tau_j) - if_3(\tau_j) + if_4(\tau_j) \right) \left( \frac{z}{\tau_j} \right)^{-n} \frac{1}{1 - \frac{z}{\tau_j}}
\]  

(26)
$$\ddot{z}\Phi'(z) + \Psi'(z) = -\frac{1}{2\pi i} \int_\gamma \left( \sigma_n - i\sigma_\ell \right) d\tau + \frac{1}{2\pi i} \int_\gamma \Phi'(-\tau) d\tau + \frac{1}{2\pi i} \int_\gamma \left( \tau - \ddot{z} \right) \Phi'(-\tau) d\tau =$$

$$= \frac{1}{2n+1} \sum_{j=0}^{2n+1} \left[ \sigma_i (\tau_j) - i\tau_{\phi} (\tau_j) \right] + \left( f_i (\tau_j) + f_2 (\tau_j) + if_3 (\tau_j) - if_4 (\tau_j) \right) \times$$

$$\left( -1 \right)^n \frac{2}{2n+1} \sum_{j=0}^{2n+1} \left( f_i (\tau_j) + f_2 (\tau_j) - if_3 (\tau_j) + if_4 (\tau_j) \right) \left( 1 - \frac{z}{\tau_j} \right)$$

$$+ \frac{2}{2n+1} \sum_{j=0}^{2n+1} \left( f_i (\tau_j) + f_2 (\tau_j) - if_3 (\tau_j) + if_4 (\tau_j) \right)$$

\( (27) \)

### 3 Numerical results for the interaction of the liner with the intact rock

This example has been drawn from a typical tunnelling application referring to a tunnel of diameter \( 2R=12 \) m excavated in a rock mass characterized by \( E = 3000 \text{ MPa}, \nu = 0.3 \), and subjected to a horizontal to vertical stress ratio equal to \( \sigma_n / \sigma_\ell = \sigma_1 / \sigma_2 = 1/2 \). The shotcrete (a form of sprayed concrete) is reinforced with steel sets that are placed at a fixed distance apart of 1 m in one case, and 0.5 m in another case. The thickness and modulus of elasticity of the composite section of the support may be found from the formulae proposed by Carranza-Torres and Dieterichs (2009). The input data for the example are summarized in Table 1. The width of the composite longitudinal section is considered to be one meter. The section is comprised of shotcrete of thickness \( h=20 \) cm and one full steel section type W6x25 along the width in the first case, and two full steel sections in the second one. The same tunnel radius, in situ stresses and mechanical properties for the ground and shotcrete as in the case represented in Figures 2, 3, and 4 will be considered here. All remaining properties for the steel set components are listed in Table 1. In the same table the equivalent thicknesses and elastic moduli of the composite shell for the two cases at hand, are also displayed.

Figures 2a, b and c present the angular distributions of the interfacial stresses acting on the elastic body for the plain and composite shotcrete sections and for the two limiting interfacial conditions for \( n=40 \) collocation points. As was intuitively expected the installation of steel-sets reduces the maximum tangential stress acting at the springline (\( \theta = 0 \text{ rad} \)) in all cases.

**Table 1** Shotcrete, steel set properties and equivalent thickness and elastic modulus of the composite sections for the two example cases.

<table>
<thead>
<tr>
<th>Shotcrete properties</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Thickness</td>
<td>0.2 m</td>
</tr>
<tr>
<td>Elastic modulus</td>
<td>3.00E+04 MPa</td>
</tr>
<tr>
<td>Poisson's ratio</td>
<td>0.3</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>W6x25 steel set properties</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Thickness of section</td>
<td>0.162 m</td>
</tr>
<tr>
<td>Area of section</td>
<td>0.004748 m²</td>
</tr>
</tbody>
</table>
Moment of inertia of section \(0.0000223\) m\(^4\)  
Elastic modulus \(2.00\times10^5\) MPa  
Poisson's ratio \(0.3\) -  

**Distance among steel sets = 1 m**  
Equivalent thickness of section \(0.205\) m  
Equivalent elastic modulus \(36525\) MPa  

**Distance among steel sets = 0.5 m**  
Equivalent thickness of section \(0.209\) m  
Equivalent elastic modulus \(40130\) MPa  

Further, the distributions of the tangential stress acting at the middle surface of the plain and composite thin liners for the two interfacial conditions are displayed in Figure 3. From these diagrams it may be noted that the tangential stress acting on the thin liner increases as the distance between the steel sets decreases for both limiting interfacial conditions; however, the tangential stress for the full-slip contact condition is larger compared to the no-slip case near the spring-line region of the tunnel, albeit it is smaller at the crown and bottom regions of the tunnel. Furthermore, the angular distributions of the dimensionless bending moment \(M/(\sigma R)\) acting on the support for the considered example cases are illustrated in Figure 4. It is remarked that the bending moment has been calculated from the approximate formula (9) in the following form

\[
M \approx \frac{h}{6} \left[ h \sigma_o (h/2) - T \right]
\]  

(28)

The dimensionless stresses that were presented above, should by multiplied by \((1 - \lambda)\) where \(\lambda\) represents the stress relief factor of the hole boundary in a pre-stressed ground. This factor varies from the zero value (no stress relief) to one (full stress relief), and depends on the distance of the installation of the support from the face of the tunnel, as well as from the stiffness of the support (that is the ratio of the pressure applied by the ground to the support over the displacement of the support) according to the convergence-confinement method of tunnel design. By putting \(\lambda = 0\) it is a conservative way of designing a support. So, in the practice of tunnel support design, the beneficial effect of the presence of the tunnel front when the liner is installed is normally accounted for by consideration of a ‘fictitious traction’ that acts at the periphery of the tunnel when installing the support (Exadaktylos and Stavropoulou 2003). In a recent publication Carranza-Torres et al. (2013) re-visited the circular lined tunnel viewing it as a two-stage process, thus accounting for the delayed installation of the support behind the face. They have finally shown that the stress reduction by the factor \((1 - \lambda)\) is a legitimate way to account for the delayed support installation effect.
Figure 2  Angular distribution of interfacial stresses acting on the elastic body for the cases of plain and composite shotcrete: (a) tangential stress, (b) radial stress, and (c) shear stress (number of collocation points = 40). The stresses are divided with $\sigma_v$. 
Figure 3  Comparison of the tangential stress distribution acting on the thin shell at its middle-surface $\sigma^t_0(0)$ for no-slip and full-slip interface conditions for the plain and composite shotcrete liners. The stresses are divided with $\sigma_v$.

Finally, the full-plane distributions of the stresses around the opening supported by the composite liner with full-slip conditions and a 0.5 m distance apart of the steel sets, are illustrated in Figures 5a, 5b and 5c, respectively. These stresses have been found by recourse to Eq's (25), (26), and (27).

Figure 4  Comparison of the dimensionless bending moment ($M / (\sigma_v R)$) distribution acting on the thin shell at its middle-surface for no-slip and full-slip interface conditions for the plain and composite shotcrete liners.
4 Concluding remarks

The contact problem of the lined circular opening in an elastic rock was formulated in terms of the complex variable theory in plane strain conditions. The liner is assumed to behave like a thin shell carrying both thrust and bending moment. It was outlined that this approach is appropriate for the consideration of the presence of cracks, joints and faults close to the opening. Here, for the brevity of presentation there are presented only results for the interaction of the intact ground with the closed ring for no-slip or full-slip conditions. In a forthcoming publication the general theory and relevant applications of the interaction of the elastic ground transected by cracks with the thin liner will be demonstrated.

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References


The use of elastic superposition as part of a multi-tiered probabilistic ground support design approach

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Abstract

Uncertainty and variability is part of geotechnical design and need to be taken into account in the design process. This paper presents some of the work in progress of the sub-project of the Australian Centre for Geomechanics’ Ground Support Optimisation Project, specifically aiming at advancing risk based support design in mining application. The paper presents some thoughts on a general multi-tiered approach to support design but focuses on its application on only one aspect of the support design process, namely the evaluation of the depth of rock mass yielding. The use of elastic superposition is discussed in more detail as one of the methods to evaluate the influence of the uncertainty of the stress state and to evaluate the depth of rock mass yielding probabilistically.

1 Introduction

Due to the uncertainty and variability associated with geotechnical design, it is necessary for geotechnical engineers to develop an appreciation of the influence of these uncertainties and variability on the reliability of the design. Probabilistic evaluation, or in the very least sensitivity analysis, should therefore form part of the design process.

This paper presents some of the work in progress as part of the Australian Centre for Geomechanics’ Ground Support Systems Optimisation Project. A section of this project aims to not only advance the state of the art, but also to advance the state of practice by providing simple and practical tools to allow mining industry practitioners to implement probabilistic ground support evaluation and to evaluate the effect of uncertainty on the design reliability.

The paper is not concerned with an exhaustive ground support design approach but will focus only on one aspect, namely, the depth of rock mass yielding which will of course impact on the decision on support length and type. A multi-tiered approach is necessary to optimise the design by balancing the quality of the results with the effort required to improve the analysis and is presented in this paper. The first level of this multi-tiered approach, namely, using elastic superposition as a tool to explore the boundaries of design problem, is presented in more detail.

2 Multi-tiered approach to assessing depth of rock mass yielding

For the purpose of this paper, the multi-tiered approach presented here is focussed at assessing the depth of rock mass yielding, but the approach can easily be extended to incorporate other aspects of the support design.

Figure 1 provides a flow chart of a systematic approach to evaluate the depth of rock mass yielding, taking account of the geotechnical uncertainty and variability inherent to the problem.
2.1 Simplified design calculations

Often, especially in early studies, little data is available and is often accompanied by a poor understanding of the rock mass. Performing simple analysis often provides some insight into the constraints of the problem. It allows the geotechnical engineer to develop an appreciation of the influence of different parameters on the outcome of the problem – knowledge that can often guide geotechnical engineers in focussing their efforts on obtaining data relevant to the problem at hand.

2.2 Data gathering and analysis

Too many design projects appear to rely on a data gathering campaign that was not properly designed and executed by people who were not aware of, or did not understand the importance and influence of obtaining data and how this data will be used in design (Hadjigeorgiou 2012). This may lead to a lot of effort spent on gathering data which may have a small influence on the design while important parameters are neglected. (Potvin et al. 2012) points out that rock mass ratings data gathered for one purpose may not be relevant to another purpose as the factors included in the rating may not be applicable to the design.

The in situ stress regime is one of the parameters often neglected and designs are often based on industry default of best guess values.

2.3 Developing probabilistic models for parameters

Having gathered field data, the data needs to be evaluated and assessed statistically; the first step towards building a probabilistic model of the parameters. Often a statistical best-fit is seen as being synonymous to developing a probabilistic model of the parameter. A statistical best-fit procedure of data does not take into account information other than the data itself and may violate physical or numeric constraints. For
example, a statistical best-fit of GSI data may result in a normal distribution. In that case it will not be the best probabilistic model as the normal distribution will model values less than 0 and more than 100. In such a case using the beta distribution to describe the GSI distribution provides a better probabilistic model for the GSI parameter.

Some assumptions like the absolute upper and lower boundary values and the type of distribution may need to be introduced to obtain a workable probabilistic model. Similar to other design assumptions, the influence of the assumptions relating to the probabilistic model, on the design outcome will need to be tested.

2.4 Design calculations and analysis

In order to optimise the design effort, one needs to balance the calculation and analysis effort with the quality of the outcomes. Refinement in calculations which will not lead to a change in a design decision is not necessary. For this reason we proposed a tiered approach with each new tier consisting of more complex methods requiring more time invested. Each of the performed analysis provide information to focus and refine the next set of calculation and maximise the return on the investment in time and effort.

In the particular case of assessing the depth of rock mass yielding around excavations, elastic superposition provides a simple but powerful approach to probabilistically evaluate the rock mass yielding depth. The elastic superposition method assumes elastic behaviour and can only be used with pure elastic analysis. This approach may be sufficient in some conditions, but in many cases one would need to step up to the next tier to perform the evaluation with elasto-plastic analysis, which may in some cases not be adequate, requiring to step up to the next tier which may require explicit modelling of anisotropy, specific discontinuities etc.

2.5 Performing sensitivity analysis and probabilistic calculations

With the method of elastic superposition it is only necessary to perform six unit analyses using numerical analysis software. The resulting stress field for any input stress state can then be calculated from the results of the unit analysis. Elastic stress superposition therefore provides a practical way to assess the stress influence of stress magnitude and orientation on a design.

The elastic superposition methods are well suited for many calculations and probabilistic calculations can be performed very easily and quickly using the Monte-Carlo method (Kroese and Rubinstein 2012), the Point Estimate Method (PEM) (Christian and Baecher 1999, Harr 1989, Rosenblueth 1975, 1981), the Response Surface Method (RSM) (Khuri and Mukhopadhyay 2010, Lü and Low 2011, Mollon et al. 2009) and the Response Influence Factor (RIF)\(^1\) method (Chiwaye and Stacey 2010, Tapia et al. 2007, Wesseloo and Read 2009).

As soon as one engages in more complex analysis methods, performing enough Monte-Carlo trials to obtain accurate results is not practical. The PEM, RSM and RIF methods, however, require a lot less analyses and are still practically achievable. The number of required analyses increases with the number of parameters for which the probabilistic model is taken into account.

Due to the fact that these methods require an increasing number of analyses with an increasing number of probabilistic parameters, it may be necessary to perform a parameter reduction analysis with the simpler methods, before embarking on probabilistic analysis using the more complex methods.

\(^1\) Due to some similarity to the Response Surface Method, the mentioned authors refer to the method as a RSM. As no response surface is derived in this method and to distinguish it from a true RSM, we refer to it as a Response Influence Factor method which is a more accurate title for the method.
The parameter reduction process involves determining the parameters which have the largest influence on the variance of the outcome variable (depth of yielding in this case), and to ignore the parameter variances which have a negligible influence on the variance of the outcome variable (Tapia et al. 2007).

3 Uncertainty in stress conditions

In many situations, the stress condition is unknown. Several stress measurement databases have been compiled for different regions and for the world (Heidbach et al. 2007, Hillis and Reynolds 2000, Stacey and Wesseloo 1998, 1999, Wesseloo and Stacey 2006). These stress databases are extremely valuable for providing an indication of the general stress level that could be expected in a region. These databases, however, also show the large degree of variation over small distances.

The local in situ stress regime can be dramatically altered by the presence of geological structure like anisotropy, faults, or intrusions. It is, therefore, important to have local stress measurements performed for the site. Even with stress measurements for a site, there is often a large degree of uncertainty in the stress regime.

An important benefit of the elastic stress superposition method is that one requires only six unit analyses to be performed and that the results for any input stress state can be calculated without having to rerun the numerical model. For this reason the elastic stress superposition is a valuable method for exploring and quantifying the influence of stress magnitudes and orientations on the problem. The method assumes linear elastic behaviour and therefore does not take load-shedding due to rock mass yielding into account.

Notwithstanding, the influence of the uncertainty in the stress regime is seldom explicitly taken into account in an analysis. If variation in the input stress regime is taken into account it generally seems to be done only for the purpose of calibration of the model to obtain the stress tensor producing the results that best match to observation (Basson and Van Der Merwe 2007, Wiles 2007). The method suggested by these mentioned authors relies on a systematic change in the input stress tensor with subsequent model runs for each input variation and can be replaced with a much more efficient use of the elastic stress superposition.

4 Elastic superimposition of stresses

4.1 Principles

The principle of elastic superposition is commonly used in mechanics calculations and forms one of the building blocks in the formulation of closed form solution in elasticity theory. Elastic stress superposition also forms an integral part of standard procedures in beam analysis, in structural engineering, where a desired standard loading condition is substituted with the sum of a set of standard loading states for which general solutions exist. The stress and displacement of the beam under the combined loading is then obtained simply as the sum of the solutions for the different stress states.

Before the advent of numerical calculations with finite-element analysis, it was often used in geotechnical engineering and tunnelling to obtain the full elastic stress state around underground openings (Morgan 1961, Terzaghi and Richart 1952).

The principal of elastic superposition can be explained as follows:

Under linear elastic conditions, a three dimensional stress state should be applied, \( S^a \), to a rock mass with an underground opening. At any point of interest in the rock mass, the resulting full three-dimensional stress state, \( S^r \), can be calculated. A different applied stress state, \( S^b \), will result in a different three-dimensional stress state, \( S^r \), at the point of interest.

The principle of superposition states that for an applied stress state \( S^{total} = S^a + S^b \), the resulting stress state at point of interest, \( S^r \), is given by the sum of the individual results, \( S^r = S^r_a + S^r_b \) (Figure 2).

Using the principle of elastic superposition, one can calculate the full three-dimensional stress field for any input stress state by superposition of the results of six unit analyses.
Method of elastic superposition

A full three-dimensional stress tensor consists of six independent components. These are the components in the three axis directions, namely $\sigma_{xx}$, $\sigma_{yy}$, $\sigma_{zz}$, and the three shear stress components in $\sigma_{xy}$, $\sigma_{yz}$ and $\sigma_{xz}$. The applied stress state, $S$, can be written as a tensor as follows:

$$ S = \begin{bmatrix} \sigma_{xx} & \sigma_{xy} & \sigma_{xz} \\ \sigma_{xy} & \sigma_{yy} & \sigma_{yz} \\ \sigma_{xz} & \sigma_{yz} & \sigma_{zz} \end{bmatrix} $$

The stress state, $S$, can be written as a superposition of six unit stresses as follows:

$$ S^{\text{total}} = \sigma_{xx}S^x + \sigma_{yy}S^y + \sigma_{zz}S^z + \sigma_{xy}S^{xy} + \sigma_{yz}S^{yz} + \sigma_{xz}S^{xz} = \begin{bmatrix} \sigma_{xx} & \sigma_{xy} & \sigma_{xz} \\ \sigma_{xy} & \sigma_{yy} & \sigma_{yz} \\ \sigma_{xz} & \sigma_{yz} & \sigma_{zz} \end{bmatrix} $$

where each of the units stresses is given as:

$$ S^x = \begin{bmatrix} 1 & 0 & 0 \\ 0 & 0 & 0 \\ 0 & 0 & 0 \end{bmatrix}, \quad S^y = \begin{bmatrix} 0 & 0 & 0 \\ 0 & 1 & 0 \\ 0 & 0 & 0 \end{bmatrix}, \quad S^z = \begin{bmatrix} 0 & 0 & 0 \\ 0 & 0 & 1 \\ 0 & 0 & 0 \end{bmatrix} $$

For each of these six unit analyses ($c = xx, yy, zz, xy, yz, xz$) a full three-dimensional stress field can be obtained:

$$ S^c = \begin{bmatrix} s^c_{xx} & s^c_{xy} & s^c_{xz} \\ s^c_{xy} & s^c_{yy} & s^c_{yz} \\ s^c_{xz} & s^c_{yz} & s^c_{zz} \end{bmatrix} $$

For an input stress state of $S^{\text{total}}$, the resulting stress field can then be calculated as the superposition of the results from each of the unit stress analyses, i.e.
The use of elastic superposition as part of a multi-tiered probabilistic ground support design approach

S_r = \sigma_{xx}S_r^x + \sigma_{yy}S_r^y + \sigma_{zz}S_r^z + \sigma_{xz}S_r^{xz} + \sigma_{yz}S_r^{yz} + \sigma_{zx}S_r^{zx} = \\
\sum_c \sigma_c \cdot S_r^x \sum_c \sigma_c \cdot S_r^y \sum_c \sigma_c \cdot S_r^z \\
\sum_c \sigma_c \cdot S_r^{xy} \sum_c \sigma_c \cdot S_r^{yx} \sum_c \sigma_c \cdot S_r^{yz} \\
\sum_c \sigma_c \cdot S_r^{xz} \sum_c \sigma_c \cdot S_r^{zx} \sum_c \sigma_c \cdot S_r^{xy}

5 Probabilistic evaluation of depth of rock mass yielding using elastic stress superposition – example application

This paragraph briefly presents an example application of the use of the elastic stress superposition method to perform a probabilistic assessment of the depth of rock mass yielding. Six unit analyses were performed using Map3D Fault Slip v64 (Wiles 2015) and the results of each of these unit analyses were exported.

Post processing was performed in a custom app built in mXrap (Harris and Wesseloo 2015). The value of the method for “what-if” type sensitivity analyses becomes clear when one considers the fact that each one of the unit analyses ran for several hours, while the post-processing calculated the results for a new input stress on several planes within a fraction of a second. This calculation efficiency has obvious benefit for probabilistic calculations.

For the purpose of this analysis, the intact rock parameters, m_i, and UCS and the rock mass quality parameter GSI were assumed to be independent. A Hoek-Brown failure criterion was used to model the rock mass strength envelope. The Hoek-Brown rock mass parameter was based on the formulation by (Carter et al. 2008) who extended the Hoek-Brown failure criterion to be applicable also to spalling and squeezing conditions.

Figure 3 shows the strength factor plots resulting from the post processing analysis for three different scenarios. The strength factor at each evaluation point was calculated as the stress to strength ratio at each point. Each one of these scenarios had identical parameters to the mean parameter values shown in Table 1, except for the value of \( \sigma_1 \) and the value of the dip of \( \sigma_1 \) as noted in the Figure 3.

Considering the uncertainty that often exists with respect to the local in situ stress state, the example highlights the importance of taking the stress state uncertainty into account in the design process.

<table>
<thead>
<tr>
<th>Table 1</th>
<th>Table of input parameters</th>
</tr>
</thead>
<tbody>
<tr>
<td>GSI</td>
<td>Truncated normal</td>
</tr>
<tr>
<td>m_i</td>
<td>Truncated normal</td>
</tr>
<tr>
<td>UCS</td>
<td>Truncated normal</td>
</tr>
<tr>
<td>( \sigma_a )*</td>
<td>Normal</td>
</tr>
<tr>
<td>( \sigma_b )</td>
<td>Normal</td>
</tr>
<tr>
<td>( \sigma_c )</td>
<td>Normal</td>
</tr>
<tr>
<td>( \sigma_a ) dip</td>
<td>Normal</td>
</tr>
<tr>
<td>( \sigma_c ) dip dir.</td>
<td>Normal</td>
</tr>
<tr>
<td>( \sigma_c ) rake</td>
<td>Normal</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
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<th>Mean</th>
<th>Stdev</th>
<th>Lower limit</th>
<th>Upper limit</th>
</tr>
</thead>
<tbody>
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<td>10</td>
<td>0</td>
<td>100</td>
</tr>
<tr>
<td>m_i</td>
<td>10</td>
<td>2</td>
<td>5</td>
<td>15</td>
</tr>
<tr>
<td>UCS</td>
<td>90</td>
<td>15</td>
<td>50</td>
<td>200</td>
</tr>
<tr>
<td>( \sigma_a )*</td>
<td>50</td>
<td>5</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>( \sigma_b )</td>
<td>50</td>
<td>5</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>( \sigma_c )</td>
<td>20</td>
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<td>-</td>
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<td>45</td>
<td>10</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>( \sigma_c ) dip dir.</td>
<td>0</td>
<td>10</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>( \sigma_c ) rake</td>
<td>0</td>
<td>10</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>

* Since the distributions for the different principal stresses overlap \( \sigma_a \), \( \sigma_b \), and \( \sigma_c \) is specified and not \( \sigma_1 \), \( \sigma_2 \), and \( \sigma_3 \).
Figure 3  Strength factor calculated for three different stress states using stress superposition post-processing
The use of elastic superposition as part of a multi-tiered probabilistic ground support design approach  

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Figure 4  Equi-probability contours of yielding depth using stress superposition post-processing

Figure 4 displays the equi-probability contours on chosen analyses planes. For every point on the planes of interest, the probability of that point being in a state of yielding is calculated for each of the Monte-Carlo runs. Combining the results for each of the Monte-Carlo runs results in the probabilistic depth of yielding calculations.

The probability contours are calculated by performing random deviate sampling from each of the input distributions (both the stress and strength parameters). For each sample sets, the elastic superposition is performed to obtain the resulting stress field. The strength factor at each point is calculated for the sample stress parameters and the probability of yielding at every evaluation point is taken as the ratio of Monte-Carlo trials that yielded to the total number of trials.

Adjacent to the boundary of the excavation, the contours indicate a 90-100% probability, indicating that this region yielded in more than 90% of the Monte-Carlo trials. Based on this assessment, we therefore have a 90% confidence that this zone will be contained within the yielding zone. In other words, the probability of the yielding zone exceeding the depth indicated by the 90% contour line is 90%. The 20% contour line indicated the depth for which we have an 80% confidence that it will not be exceeded.

Comparison of Figures 3 and 4 shows that the depth of yielding calculated for the mean conditions corresponds roughly with a depth of yielding with a 70% probability of being exceeded. It should be noted that the depth of yielding with a 10% probability of being exceeded is about three times the depth estimated from mean parameters.

6  Concluding remarks

This paper presented a multi-tiered approach to assessing the depth of rock mass yielding. This multi-tiered approach can easily be applied to other aspects of support design. Geotechnical design is subject to many uncertainties, some of which can be large. Uncertainty in the stress regime is one of those components, which is seldom taken into account in a systematic way. The application of elastic stress superposition provides an efficient method to explore the influence of different input stress states on the excavation stability. Its computational efficiency also allows one to explore the sensitivity of the outcome to assumptions on the input parameters and their distributions. The elastic stress superposition method is limited to elastic analysis and can therefore only be part of the first tiers of the multi-tiered approach. The example application shows the importance of the probabilistic evaluation as it highlights the large uncertainty in the yielding depth that may occur, a component of support design seldom explicitly and systematically taken into account.
Acknowledgement

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Theoretical investigation of the effect of stress on the performance of support systems based on Rock Mass Rating (RMR) support recommendations

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Hakan Başarır, The university of western Australia, Australia
Johan Wesseloo, Australian centre for geomechanics, The university of western Australia, Australia

Abstract

The Rock Mass Rating (RMR) classification system is widely used to design support systems for underground openings. For underground openings, excavated in the same quality rock mass, the system proposes the same support system independent from the depth of the opening. This study theoretically analyses the performance of RMR proposed support for openings excavated in the same quality rock masses at different depths.

A very comprehensive numerical modelling programme was conducted. Different case studies were collected from current literature, presenting the results of the underground support design projects conducted by different researchers. The quality of the rock masses range from poor to fair. Using the collected information, numerical models of selected cases were constructed and internal support pressure equivalent to RMR proposed support systems was applied. Next, the in situ stress conditions were changed by assigning different depths and corresponding displacements were calculated. A critical strain criterion was used to evaluate the relative performance of the RMR proposed support systems. The support pressure necessary for keeping the strain below critical strain values was found iteratively for each case study, under changing stress conditions. Finally, an empirical equation and graph were produced showing the relationship between RMR, depth and the support pressure ratio, obtained by dividing necessary support pressure by RMR proposed support pressure.

It was concluded that the pressure supplied by the RMR system is adequate for the openings excavated in good quality rock masses and rock masses at shallow depth. However, as the rock quality decreases and the depth increases, the support pressure provided by the proposed support system may not be adequate for maintaining the opening’s stability.

KEYWORDS: Rock Mass Rating (RMR) system, rock mass classification, empirical design, numerical modelling, critical strain, support system performance evaluation.

1 Introduction

Due to the complex geological and geotechnical environment, the design of underground mining openings such as drifts and shafts is always considered as a difficult problem. For many years empirical methods have provided a practical solution to these problems and they have been used widely for the assessment of the support requirements to ensure stability.

One widely used design method is the RMR system proposed by Bieniawski (1974). The system was developed using a database mainly composed of tunnels excavated in sedimentary rocks at relatively shallow depths and, therefore, relatively low stress conditions. In 1989, the system was updated and the database was expanded (Bieniawski, 1989) by adding 351 different case studies.
The input parameters of the system are: uniaxial compressive strength of the rock material (UCS), rock quality designation (RQD), joint spacing, joint conditions and groundwater conditions. Different values for each of the parameters are assigned and the overall basic rating is obtained by summing up the individual components’ weightings. Corrections are applied to basic RMR values to take into account the effect of joint set orientation relative to excavation direction. Based on this final value, the support system is proposed for the underground opening (Bieniawski, 1989). Although the system is widely used, some researchers proposed different corrections such as stress correction, blasting adjustment, and weakness plane consideration as mentioned by Ulusay and Sonmez (2007).

Apart from rock mass properties, which are accounted for in the RMR system, the opening stability is also governed by the opening geometry and stress conditions that are not accounted for in the RMR system and, therefore, not incorporated in the support recommendations. Since the support recommendations proposed by the RMR system are based on a database dominated by relatively shallow excavations, the question arises as to how different stress conditions would affect the performance of the suggested support.

This study theoretically investigates this question and it is limited to a comparison of support performance for a circular opening under uniform loading conditions; a simplification that needs to be addressed in further studies.

2 Methodology

The methodology used in this study is briefly described as follows:

- Rock mass strength and deformability properties were calculated, as suggested by Hoek et al. (2002), based on the geological strength index (GSI) values (Hoek et al. 1995), Hoek-Brown constant (\(m_i\)) and unconfined compressive strength (UCS).
- Calculation of equivalent support pressure, provided by RMR proposed support systems (PRMR), using equations suggested by Carranza-Torres and Fairhurst (2000).
- Numerical evaluation of tunnel strain, with varying support pressure and depth.
  - For depths ranging from 50 to 1,000 m the calculated support pressure was applied as an active internal pressure in the model and resulting displacement and tunnel strain were calculated for different stress conditions.
  - The necessary support pressure \(P_e\), to limit the total tunnel strain to 2%, was determined iteratively for different depths.

Analyses were performed for a range of rock mass conditions. The specific rock mass conditions used in the study were those corresponding to that encountered at three tunnels, for which the ground support was based on the suggested support from the RMR system (Appendix A).

3 Case studies and rock properties

The collected case studies are composed of three roadway tunnels (Ghafoori et al. 2006; Sari et al. 2008; Satıcı, 2007). The strength and deformability properties calculated using the information presented in these studies were used as input for a two-dimensional finite element method based software. For most cases, GSI values were supplied. For the cases where GSI values were not available, they were estimated using the suggested equation \(\text{GSI} = \text{RMR}_{89} - 5\) (Hoek and Brown, 1997), where \(\text{RMR}_{89}\) was 1989 version of Bieniawski’s RMR classification (Bieniawski, 1989) calculated by setting the groundwater rating to 15 and the adjustment for joint orientation to zero. A comprehensive review of the relationship between RMR and GSI was presented by Osgoui and Ünal (2005).

The first case, Osmangazi tunnel, is located in the Bilecik province of Turkey. The width, height and length of the tunnel are 12.5, 9.6 and 2500 m, respectively. Seven different rock masses were observed through
the route of the Osmangazi tunnel (Sari et al. 2008). Kallat highway tunnel is the second case, located in the Masshad province of Iran. The width, height and length of the Kallat tunnel are 8, 8.4 and 725 m, respectively. Three different rock masses were encountered through the Kallat tunnel. The last case study is the Sehzadeler highway tunnel, located in the Amasya province of Turkey. Its width, height and length are 12, 9, 345 m, respectively. There were four different rock masses observed in the Sehzadeler tunnel. Rock mass and material properties such as GSI, RMR, intact rock strength (UCS), intact rock deformation modulus (E<sub>i</sub>), Hoek Brown constant (m<sub>i</sub>), Poisson’s ratio (ν), and unit weight (γ) of the rock units observed through the tunnels are given in Table 1. The depth of the Osmangazi, Kallat and Sehzadeler tunnels ranges from 30 to 280, 40 to 160 and 30 to 100 m, respectively.

Table 1 Rock mass and material properties of rock units observed along tunnel route

<table>
<thead>
<tr>
<th>Case</th>
<th>Rock unit</th>
<th>RMR</th>
<th>GSI</th>
<th>UCS, MPa</th>
<th>E&lt;sub&gt;i&lt;/sub&gt;, MPa</th>
<th>m&lt;sub&gt;i&lt;/sub&gt;</th>
<th>ν</th>
<th>γ, MN/m³</th>
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<tbody>
<tr>
<td>Osmangazi tunnel (Sari et al. 2008)</td>
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<td></td>
<td></td>
</tr>
<tr>
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<td>27</td>
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<td>3976</td>
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<td>0.25</td>
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<td>90</td>
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</tr>
<tr>
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</tr>
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<td>60</td>
<td>85</td>
<td>7703</td>
<td>18</td>
<td>0.31</td>
<td>0.31</td>
<td></td>
</tr>
<tr>
<td>Kallat tunnel (Ghaforri et al. 2006)</td>
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<td></td>
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</tr>
<tr>
<td>Sehzadeler tunnel (Satici, 2007)</td>
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<td>10000</td>
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<td>0.30</td>
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</table>

4 Numerical modelling

In this study, a two-dimensional finite element method based software, Phase2, was used. By means of a prepared patch, rock properties and opening geometries can easily be changed. Moreover, the corresponding tunnel displacement readings can be recorded automatically. For each rock unit, unsupported and supported cases were analysed by applying different stress conditions in terms of depths. In total, 462 runs were conducted. The calculation of rock properties, prediction of internal support pressure, model geometries, stress conditions and the assessment of necessary support pressure for retaining opening stability are the main components of numerical modelling, as explained below.

4.1 Rock properties

The mechanical properties of the rock masses were calculated using the equations suggested by Hoek et al. (2002), as they would be used as input parameters for numerical analysis. The rock units observed through the routes of the tunnels and their strength and deformability properties such as rock mass deformation modulus (E<sub>m</sub>), Hoek Brown constants (m<sub>b</sub>, s<sub>m</sub>), are presented in Table 2.
Table 2  Calculated rock mass properties to be used in numerical analysis

<table>
<thead>
<tr>
<th>Case</th>
<th>Rock unit</th>
<th>$E_m$ MPa</th>
<th>$m_b$</th>
<th>$s_m$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Osmangazi tunnel (Sari et al. 2008)</td>
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<td>0.0019</td>
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<table>
<thead>
<tr>
<th>Case</th>
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<th>$m_b$</th>
<th>$s_m$</th>
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</thead>
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<tr>
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<table>
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<th>$E_m$ MPa</th>
<th>$m_b$</th>
<th>$s_m$</th>
</tr>
</thead>
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<td>790</td>
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</table>

4.2  Model geometry and calculation of internal support pressure and critical strain

As the opening geometries are different, the equivalent radius approach allowing the use of circular shape for the openings with different shapes (Curran et al. 2003) was used. The calculated equivalent diameters for the Osmangazi, Kallat and Sehzadeler tunnels are 11.24 m, 9.03 m and 10.20 m, respectively.

The equivalent support pressures of the RMR proposed support systems were calculated using the method proposed by Carranza-Torres and Fairhurst (2000). The calculated support pressures were applied to the opening as internal support pressure. The used equations are listed below. For each depth, the calculated support pressure of the RMR proposed support system was applied and corresponding displacements were calculated.

\[
P_{sh}^{\text{max}} = \frac{\sigma_{cc}}{2} \left[ 1 - \frac{(R-t_c)^2}{R^2} \right]
\]

where:

- $P_{sh}^{\text{max}}$: maximum support pressure of shotcrete, MPa.
- $\sigma_{cc}$: uniaxial compressive strength of shotcrete, MPa.
- $t_c$: shotcrete thickness, m.
- $R$: outer radius of support, m.

\[
P_{b}^{\text{max}} = \frac{\tau_{bf}}{s_c s_l}
\]

where:

- $P_{b}^{\text{max}}$: maximum support pressure of rockbolt, MPa.
- $\tau_{bf}$: maximum carrying capacity of rockbolt, MPa.
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4.3 Stress conditions

The calculation of the horizontal stresses is a difficult task, especially for shallow tunnels. Hoek and Brown (1980) analysed in situ measurements around the world and concluded that horizontal stresses vary at shallow depth, whereas they tend to be closer to equal to the vertical stresses in deep environments. For simplicity’s sake, similar to the previous studies (Asef et al. 2000; Sari, 2007; Basarir, 2008; Basarir et al. 2010), in this study hydrostatic stress conditions were applied. The vertical stress is calculated as the overburden stress.

This simplifying approach may not be appropriate for support design in general. In this study, however, we are performing a comparative study. Future extension of this analysis should take the stress ratio into account.

4.4 The prediction of support performance and the pressure necessary for keeping opening stability

In order to quantify the effect of different stress conditions on the performance of the proposed support systems, a robust and dimensionless measure of support performance is necessary. Sakurai (1983) suggested the use of tunnel strain, defined as the ratio between opening width and observed displacement, as a stability indicator. Hoek (2000) suggested that when the strain value of 2% is exceeded, the stability of the opening cannot be maintained. The 2% criterion is a questionable design criterion, but for the purpose of comparison it provides a robust and simple measure to compare the support performance under different stress conditions.

In this study, the total opening was strain defined as the maximum elasto-plastic displacement normalised to equivalent tunnel diameter.

For the considered highway tunnels, the 2% critical strains were calculated and corresponding limiting displacements were determined as 0.23, 0.18 and 0.20 m for the Osmangazi, Kallat and Sehzadeler tunnels, respectively. The support pressure necessary for keeping the opening strain or limiting deformation less than calculated was found iteratively and recorded as required support pressure (Pg). The flowchart showing the steps followed to obtain Pg is shown in Figure 1.
Theoretical investigation of the effect of stress on the performance of support systems.

E. Karakaplan, H. Basarir, J. Wesseloo

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Figure 1  The flowchart followed for the determination of required support pressure (Pg)

The graph showing the relationship between depth and critical strain for the case studies is given in Figure 2.

Figure 2  The relationship between depth and strain for RMR suggested support systems for the case studies

5  Analysis of numerical modelling results

The graph showing the relationship between depth and required support pressure for keeping the strain less than 2% for the rock masses with different qualities is shown in Figure 3. As the quality of rock mass increases, the required support pressure decreases. For the same rock mass quality as the depth increases the required support pressure also increases. The rate of the increment changes depending on the quality of the rock mass.
To embrace the rock masses not included in the numerical analysis, a multiple regression modelling technique was employed. In regression modelling, the dependent variable is the ratio of required support pressure to RMR proposed support pressure ($P_{g}/P_{RMR}$). Independent variables are specified as RMR and depth. As it can be understood from the high multiple coefficient of determination ($R^2$) 88.27% obtained, the model established a valid relationship between dependent and independent variables. In Equation 4, $a$, $b$ and $c$ are constants and the values of these constants are 0.065928, 0.889308 and 1.524772, respectively. The multiple regression model also implies that the depth has a strong effect on the support ratio as the largest constant is related to the depth ($H$).

$$\frac{P_{g}}{P_{RMR}} = ab^{RMR}H^c$$  (4)

The contour map drawn by using the derived regression equation is shown in Figure 4. The case studies are also shown in Figure 4. As it can be seen from Figure 4, $P_{g}/P_{RMR}$ values for the case studies are lower than or around 1 (indicated using dashed line). This shows that for the case studies the support pressure provided by the RMR proposed support system are equal to or higher than the necessary support pressure required for keeping the strain less than 2%.

Figure 3  Necessary support pressure to keep the stability of the opening excavated in different quality rock masses and depths
6 Conclusions and recommendations

In this study the performance of the support systems proposed by the RMR classification system is evaluated theoretically by numerical modelling. The opening or tunnelling strain concept proposed by Sakurai (1983) and Hoek (2000) was used as a performance indicator for RMR proposed support systems.

For the analysis of the performance of RMR proposed support systems, a two-dimensional finite element method based software was used (Rocscience, 2009). The results obtained from finite element modelling were used for the construction of multiple regression models to cover the rock masses not included in the case studies. When the contour map drawn by using developed regression model was considered the following conclusions were derived.

The pressure provided by the RMR proposed support system seems to be adequate for the openings excavated at shallow depth and in good quality rock masses. As the quality of rock mass decreases and the depth increases, the proposed support system may not be adequate to ensure excavation stability.

Considering the case studies used, it can be concluded that depth or stress conditions should be considered together with the rock properties. Therefore, for major projects, in addition to the use of empirical methods such as the RMR system, the use of numerical modelling to check the performance of the proposed support system is strongly recommended.

In this study, in order to make a systematic approach, hydrostatic stress conditions were assumed; similar to other studies presented in the current literature. This approach can be considered as adequate for preliminary analysis, whereas for detailed and major projects the stresses should be measured in field and used in the analysis. An equivalent diameter approach is used in this study and it was assumed that in circular openings excavated under hydrostatic stress conditions, support elements were loaded symmetrically and there was not any bending moment acting on them. Whereas in reality, support elements like shotcrete and steel sets can be loaded asymmetrically and may be affected by bending moment due to surface roughness. Therefore, in practice the support pressure can be higher than those presented in this study. Due to the selected failure criterion, it was assumed that the rock mass does not contain any dominant discontinuity that can lead to anisotropic behaviour or structural instability.
For the considered case studies presented in the current literature, it was seen that the support pressure ratio \( (P_t/P_{\text{RMR}}) \) value was around or lower than 1. In other words, for the considered case studies the support pressure produced by RMR suggested support systems are close to or higher than the required support pressure. To the author’s knowledge for the case studies used in this study, there was not any stability problem. This is compatible with the results obtained from this simplified methodology for these cases, and the results of this methodology, although simplified, appear reasonable.

**References**


Appendix A  
Support system proposed by the RMR system for a 10m wide horseshoe opening (see notes) (after Bieniawski, 1989)

<table>
<thead>
<tr>
<th>Rock mass class</th>
<th>Excavation</th>
<th>Support</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Rock bolts (20 mm dia., fully grouted)</td>
</tr>
<tr>
<td>Very good rock I</td>
<td>Full face</td>
<td>Generally no support required except spot bolting</td>
</tr>
<tr>
<td>RMR: 81-100</td>
<td>3 m advance</td>
<td></td>
</tr>
<tr>
<td>Good rock II</td>
<td>Full face</td>
<td>Locally bolts in crown 3 m long, spaced 2.5 m with occasional wire mesh.</td>
</tr>
<tr>
<td>RMR: 61-80</td>
<td>1-1.5 m advance</td>
<td>50 mm in crown where required.</td>
</tr>
<tr>
<td></td>
<td>Complete support 20 m from face.</td>
<td>None</td>
</tr>
<tr>
<td>Fair rock III</td>
<td>Top heading and bench.</td>
<td>Systematic bolts 4 m long, spaced 1.5-2 m in crown and walls with wire mesh in crown</td>
</tr>
<tr>
<td>RMR: 41-60</td>
<td>1.5-3 m advance in top heading.</td>
<td>50-100 mm in crown and 30 mmm in sides.</td>
</tr>
<tr>
<td></td>
<td>Commence support after each blast.</td>
<td>None</td>
</tr>
<tr>
<td></td>
<td>Complete support 10 m from the face.</td>
<td></td>
</tr>
<tr>
<td>Poor rock IV</td>
<td>Top heading and bench.</td>
<td>Systematic bolts 4-5 m long, spaced 1-1.5 m in crown and walls with wire mesh.</td>
</tr>
<tr>
<td>RMR: 21-40</td>
<td>1.0-1.5 m advance in top heading.</td>
<td>100-150 mm in crown and 100 mm in sides.</td>
</tr>
<tr>
<td></td>
<td>Install support concurrently with excavation, 10 m from face.</td>
<td>Light to medium ribs spaced 1.5 m where required</td>
</tr>
<tr>
<td>Very poor rock V</td>
<td>Multiple drifts.</td>
<td>Systematic bolts 5-6 m long, spaced 1-1.5 m in crown and walls with wire mesh. Bolt invert.</td>
</tr>
<tr>
<td>RMR: &lt;20</td>
<td>0.5-1.5 m advance in top heading.</td>
<td>150-200 mm in crown, 150 mm in sides, and 50 mm on the face.</td>
</tr>
<tr>
<td></td>
<td>Install support concurrently with excavation.</td>
<td>Medium to heavy ribs spaced 0.75 m with steel lagging and forepoling if required. Close invert.</td>
</tr>
<tr>
<td></td>
<td>Shotcrete as soon as possible after blasting.</td>
<td></td>
</tr>
</tbody>
</table>

Shape: Horseshoe; Width: 10; Vertical stress <25 MPa; drilling and blasting.
Three dimensional numerical modelling of the deep Alborz tunnels with composite liners in squeezing condition

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H. Bazidehno, Amirkabir University of Technology, Iran

Abstract

The increase in transportation between cities has made it necessary to construct highway long and deep tunnels. Large deformations of surrounding media around tunnels are often encountered during excavations in rocks with squeezing conditions in long and deep tunnels. Major deformations may occur in a tunnel driven through squeezing rock. The resistance of the tunnel support (shotcrete lining, steel arches, rock anchors, etc.) to the convergence induces large rock pressure which may exceed the support’s load bearing capacity. The composite support systems such as liners consisting of shotcrete and steel sets have been used during tunnelling in difficult squeezing conditions.

The Alborz service and main tunnels excavated in Tehran-shomal freeway in the Alborz mountain range in Iran were studied in this paper. The squeezing potential of the Alborz tunnels was investigated based on different empirical approaches. Then the numerical modelling performed in this study, using the FLAC3D finite difference element programme, has made it possible to include the influence of the construction process of the service and main tunnels.

This paper uses an ‘equivalent section’ methodology for the mechanical analysis of composite supports, such as liners consisting of shotcrete and steel sets. This methodology for the design of liners is based on the construction of capacity diagrams, another established technique of structural analysis and concrete design that can be conveniently extended to the analysis of composite sections for tunnel liners. Numerical analyses are discussed to compare the results of computed and measured performance of a typical section and to find out possible optimizations of the support system adopted.

1 Introduction

The Tehran Shomal highway project in Iran is a new freeway in order to connect the capital Tehran with the city of Chalus at the Caspian Sea in the North. The total length of the highway is 121 km. The highway alignment includes more than 30 twin tunnels for Numerical Modeling 1- Support lanes. The Alborz Tunnel is the longest of these with a length of 6,400 m at an altitude of 2,400 m under Kandevan mountain (Figure 1). The Alborz tunnels include a 5.2m service tunnel located between two main horseshoe-shape tunnels with cross section height and width of 8.87 and 12.04 m respectively. The service tunnel is placed in distance 43m from each main tunnel axis and used for site investigation, drainage, ventilation and later as a service tunnel for the main tunnels during operation. The Alborz service tunnel was excavated by an open gripper hard rock TBM from Wirth Company. The maximum overburden is in the range of 800 m. However, the excavation of one of main tunnels has started by conventional drilling and blasting method. During excavation of the Alborz service tunnel by open gripper TBM, the following major extraordinary difficulties have been encountered: methane gas, squeezing ground and blocked cutter head and TBM shield, multiple fault zones with large crown instabilities, karst with water in gypsum / anhydritic rock, water ingress.
In this paper, squeezing hazard of the Alborz service tunnel was investigated in some parts of the tunnel with high overburden and low rock mass strength based on different empirical approaches. Then the numerical modelling performed in this study, using the FLAC\textsuperscript{3D} finite difference element programme, has made it possible to include the influence of the construction process of the service and main tunnels.

This paper uses an ‘equivalent section’ methodology for the mechanical analysis of composite supports, such as liners consisting of shotcrete and steel sets. This methodology for the design of liners is based on the construction of capacity diagrams, another established technique of structural analysis and concrete design that can be conveniently extended to the analysis of composite sections for tunnel liners.

2 Geological setting of the Alborz tunnel

Iran is located at the western tip of Alpine-Himalayan system and geological-structurally an integral part of it. The Alborz Mountains of Iran are part of Alpine-Himalayan system. The predicted geological conditions of the Alborz tunnel are complex and overall heterogeneous. In the north, Triassic and Jurassic argillites with some sandstones and thin coal layers of Shemshak formation were expected, followed by a sandstone and then limestone formation. At TM $\sim$3300-3600, a 300 m thick fault zone was predicted, representing the Kandovan fault zone. Further south, Oligocene clastic sediments (Kandovan Shale) were predicted, including massive gypsum / anhydrite bodies with a length up to 300 m on tunnel level. At the surface the gypsum shows massive karstic features with unknown extend below surface (overburden $\sim$600 m above tunnel level). The remainder of the tunnel is Eocene tuffs, shales and other layered rocks of Karaj Formation. Figure 2 shows the geological profile along the Alborz tunnel.
2.1 Squeezing hazard evaluation in the Alborz tunnel

The probability of squeezing hazard was investigated by empirical approaches. The methods used to predict squeezing hazard were summarized in Table 1. The maximum squeezing hazard is related to TM ~3300 to 3500 where the overburden is high and the surrounding rock mass is fault zone.

Table 1 Semi empirical methods to calculate and predict squeezing condition

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>squeezing parameter:</td>
<td>squeezing parameter:</td>
<td>squeezing parameter:</td>
</tr>
<tr>
<td>( \sigma_{cr} / P_0 )</td>
<td>tunnel strain</td>
<td>tunnel strain</td>
</tr>
<tr>
<td>High: &lt;0.4</td>
<td>Non squeezing:</td>
<td>Non squeezing:</td>
</tr>
<tr>
<td>Medium: 0.4-0.8</td>
<td>1 ( \leq \varepsilon^i_\vartheta \leq 2 )</td>
<td>1 ( \leq \varepsilon_i \leq 2.5 )</td>
</tr>
<tr>
<td>Low: 0.8-2</td>
<td>Medium: 2 ( \leq \varepsilon^a_\vartheta \leq 3 )</td>
<td>2.5 ( \leq \varepsilon_i \leq 5 )</td>
</tr>
<tr>
<td>Non squeezing: &gt;2</td>
<td>High: 3 ( \leq \varepsilon^a_\vartheta \leq 5 )</td>
<td>Very Serious: 5 ( \leq \varepsilon_i \leq 10 )</td>
</tr>
<tr>
<td></td>
<td>Too squeezing:</td>
<td>Too squeezing:</td>
</tr>
<tr>
<td></td>
<td>( \varepsilon^a_\vartheta \geq 5 )</td>
<td>( \varepsilon_i \geq 10 )</td>
</tr>
</tbody>
</table>

where, \( \sigma_{cr} \) is uniaxial rock mass compressive strength, \( P_0 \) is initial in situ stress, \( \varepsilon_\vartheta \) is tangential strain and \( \varepsilon_t \) is tunnel strain.

In Figure 3, the squeezing hazard potential was evaluated along the Alborz service tunnel (TM 3000~4000) based on Jethwa & Singh.(1984) approach.
Three dimensional numerical modelling of the deep Alborz tunnels with composite liners in squeezing condition

H. Molladavoodi and H. Bazidehno

Figure 3  Squeezing hazard according to Jethwa & Singh.(1984) approach.

Also the squeezing hazard potential was investigated along the Alborz service tunnel according to Aydan et al. (1993) method as shown in Figure 4.

Figure 4  Squeezing hazard evaluation according to Aydan et al. (1993) approach.

Furthermore, Figure 5 represents the squeezing hazard potential investigated along the Alborz service tunnel based on Hoek & Marinos(2000) method.

Figure 5  Squeezing hazard evaluation based on Hoek & Marinos (2000) approach.

According to Figs 3-5 & Table 1, the potential of squeezing hazard in TM (3350~3450) is very high because of high overburden and weak rock mass quality in this area. Therefore, this area was selected for the numerical simulation.
3 Ground-support interaction

Support designing needs evaluation of the loads that ground transmits to the support. In this paper, ground loads on the liner were determined by using the ground-support interaction analysis performed with a FLAC® numerical package. Because of squeezing or ravelling condition, steel arches are commonly used in combination with sprayed concrete or shotcrete. An important question related to a composite liner consisting of shotcrete and steel sets is how much bending moment and thrust is sustained by each component of shotcrete and steel sets. In order to analyze the composite liner in the Alborz tunnel, the equivalent section and capacity diagrams were implemented.

3.1 Equivalent section approach

The composite liner to be analyzed is represented in Figure 6. It involves a section of composite liner of width b comprising n units of a material ‘1’ (e.g. steel sets) and n units of a material ‘2’ (e.g. shotcrete)—note that if n units of each material exist along the width b, this is equivalent to saying that the units are spaced a distance s = b/n. Each of the units ‘1’ and ‘2’ will be assumed to be characterized by compressibility coefficients $D_s$ and $D_c$ and flexibility coefficients $K_s$ and, $K_c$ respectively. The ‘compressibility’ coefficient $D$ for an arch of cross-sectional area $A$ and Young’s modulus $E$, in plane-strain conditions, is (Carranza-Torres & Diederichs, 2009)

$$D = \frac{EA}{1-\nu^2}$$  \hspace{1cm} (1)

The ‘flexibility’ coefficient $K$ for the section in plane-strain conditions results

$$K = \frac{EI}{1-\nu^2}$$  \hspace{1cm} (2)

where $\nu$ is the Poisson’s ratio and I is the moment of inertia of the material.
The composite section in Figure 6a can be regarded as an equivalent section of width $b$ and thickness $t_{eq}$ as represented in Figure 6b. The thickness $t_{eq}$ will be computed from the coefficients $D_c$ and $D_s$, and $K_c$ and $K_s$ of the two units as follows (Carranza-Torres & Diederichs, 2009):

$$ t_{eq} = \sqrt{\frac{12 k_c + k_s}{D_c + D_s}} $$

(3)

With the thickness of the equivalent section defined by Eq. (3), the equivalent Young’s modulus $E_{eq}$ can be calculated as following (Carranza-Torres & Diederichs, 2009):

$$ E_{eq} = \frac{n(D_c + D_s)}{bt_{eq}} $$

(4)

Input data for analysis of the composed liner in the Alborz tunnel is presented in Table 1.
Table 1. Input data for analysis of the composed liner in the Alborz tunnel.

<table>
<thead>
<tr>
<th>Shotcrete</th>
<th>Steel arches IPE180</th>
</tr>
</thead>
<tbody>
<tr>
<td>tc: 0.25 m</td>
<td>ts: 0.18 m</td>
</tr>
<tr>
<td>Ec: 30 GPa</td>
<td>As: 2.39×10^-3 m²</td>
</tr>
<tr>
<td>νc: 0.15</td>
<td>Is: 1.32×10^-5 m⁴</td>
</tr>
<tr>
<td>σccc: 40 MPa</td>
<td>Es: 200 GPa</td>
</tr>
<tr>
<td>σct: -4 MPa</td>
<td>st: -500 MPaσ</td>
</tr>
<tr>
<td>σc: 0.15</td>
<td></td>
</tr>
<tr>
<td>σsc: 500 MPaσ</td>
<td>s: 0.25v</td>
</tr>
</tbody>
</table>

In one meter across the Alborz tunnel in TM (3350~3450) with squeezing condition, the spacing of steel arches are 0.5 m (s = 0.5 m). So the steel arches number per meter is \( n = \frac{b}{s} = \frac{1}{0.5} = 2 \). The ‘compressibility’ coefficient of steel arches and shotcrete can be calculated as following

\[
D_s = \frac{E_c A_s}{(1 - \nu_s^2)} = 5.1 \times 10^3 \text{ MN} \\
D_c = \frac{E_c A_c}{(1 - \nu_c^2)} = 3.762 \times 10^3 \text{ MN}
\]

The ‘flexibility’ coefficient of steel arches and shotcrete can be calculated

\[
K_s = \frac{E_c I_s}{(1 - \nu_s^2)} = 2.816 \text{ MN m}^2 \\
K_c = \frac{E_c I_c}{(1 - \nu_c^2)} = 19.58 \text{ MN m}^2
\]

The cross section area and interia moment of shotcrete can be estimated

\[
A_c = \frac{1}{n} b t_c - A_s = 0.12261 \text{ m}^2 \\
I_c = \frac{1}{n} \frac{b t_c^3}{12} - I_s = 6.38 \times 10^{-4} \text{ m}^4
\]

The equivalent ‘compressibility’ and ‘flexibility’ coefficients are calculated according to equivalent section approach

\[
D_{eq} = n(D_s + D_c) = 8.544 \times 10^3 \text{ MN} \\
K_{eq} = n(K_s + K_c) = 44.792 \text{ MN m}^2
\]

The thickness of the equivalent section and the equivalent Young’s modulus \( E_{eq} \) can be calculated

\[
t_{eq} = \sqrt{\frac{12(K_c + K_s)}{(D_c + D_s)}} = 0.2582 \text{ m} \\
E_{eq} = \frac{n(D_c + D_s)}{b t_{eq}^2} = 34064 \text{ Mpa}
\]

4 Numerical simulation of the Alborz tunnel

The Alborz service tunnel in TM (3350~3450) with squeezing rock was selected for numerical simulation with FLAC. The mechanized excavation method with TBM in the Alborz service tunnel was simulated. This part of the Alborz service tunnel was excavated in a thick weak zone consists of schist, shale, etc that is
fairly weak from the strength point of view. Moreover, the overburden height along the driven length of the tunnel reaches to 800 m at this section. It is believed that, weak strength properties and high overburden, leading to high stresses, are the major causes of ground squeezing in this area. The Alborz tunnel passed through this rock mass at an average depth of 800 m, which means that the vertical in situ stress will be $p_o = 21.8$ MPa. The usual assumption for very weak rock masses is that they are incapable of sustaining significant differential stresses and that failure occurs until the in situ horizontal and vertical stresses have been equalized (Hoek, 2001).

### 4.1 Geometry and boundary condition

For simulation of the Alborz tunnels, a numerical model was made with dimensions of 115 m width, 30 m length and 75 m height in FLAC$^{3D}$ environment. Outer boundaries are placed in far distance of the tunnel diameter till boundary effects on the numerical model results are minimized.

![Figure 7](image.png)

**Figure 7** A view of geometry and boundary condition of the model.

In order to simulate the Alborz tunnel at depth of 800 meters from the ground according to Figure 7, the velocity was selected zero in the floor, front, back, right and left boundaries of the model. The in situ vertical stress $\sigma_v = 21 MPa$ was applied to the upper boundary due to 800 meters rock overburden with 2600 kg/m$^3$ density. In order to increase the accuracy and speed, dimension of used grids increases with distance from the tunnel.

### 4.2 Strain softening constitutive model

Based on a wide experience in the field of underground rock engineering, Hoek and Brown (1997) suggest that average quality rock masses behave in a strain softening mode in post-peak region, whereas soft rock masses behave in a ductile way and hard rock masses in an elastic-brittle fashion. The pos-peak rock mass behaviour depends on the rock mass quality (GSI) as shown in Figure 8.
Figure 8  Post-peak rock mass behaviour dependent on GSI (Alejano et al. 2010).

In plastic zone, it is supposed that strength parameters of rock mass decrease by bilinear function according to a softening parameter in comparison with the critical softening parameter in the softening region and it reaches to a minimum constant value in the residual region. The critical softening parameter controls the transition between the softening and residual stages. The peak and residual values of strength parameters can be estimated from relations proposed by Hoek et al. (2002) based on geological strength index (GSI) and intact rock strength parameters. The peak values of strength parameters can be estimated from $GSI_{\text{peak}}$ determined from the field geotechnical investigation. The residual values of strength parameters can be estimated from $GSI_{\text{resd}}$. In this study based on geological strength index (GSI=40) in the studied area, Mohr-Coulomb strain softening model was implemented in numerical simulation with following parameters.

### Table 2  Constitutive model parameters in the Alborz tunnel.

<table>
<thead>
<tr>
<th>peak</th>
<th>residual</th>
</tr>
</thead>
<tbody>
<tr>
<td>GSI:40</td>
<td>GSI: 26.47</td>
</tr>
<tr>
<td>$E$: 3.98 GPa</td>
<td>$E$: 1.82 GPa</td>
</tr>
<tr>
<td>cohesion: 2.91 MPa</td>
<td>cohesion: 2.17 MPa</td>
</tr>
<tr>
<td>friction: 21.48</td>
<td>friction: 17.45</td>
</tr>
<tr>
<td>dilation: 2.68</td>
<td>dilation: 2.18</td>
</tr>
</tbody>
</table>

4.3  The Alborz service tunnel excavation and support installation

After initial geometry and in-situ field stresses of the region, the analysis was done in several steps with the elastic model until equilibrium before the tunnel excavation. The model equilibrium before excavation was controlled with unbalanced force diagram. After initial equilibrium, the displacements were set to zero in this stage. Then the Mohr-Coulomb strain softening constitutive model was called and the circular service tunnel was made null (excavated). In order to simulate the mechanized TBM excavation of the Alborz tunnel in FLAC$^3D$, a 2 m tunnel advance was taken into account in numerical simulation. After each advance excavation, a composed liner with equivalent section of thickness $t_{\text{eq}}$ and Young’s modulus $E_{\text{eq}}$ as represented in Table 1 was installed. Ground vertical stresses applied on the composed liner with equivalent parameter can be shown in Figure 9.
In Figure 9, the sequential advances of tunnel excavation with TBM can be observed. Induced displacement contours due to excavation after support installation are presented in Figure 10.

Maximum displacement due to excavation and after composed liner installation is about 4.5 cm as shown in Figure 10. A plastic zone is usually formed around tunnels with squeezing condition. Figure 11 shows the plastic zone occurred surrounding the Alborz service tunnel with support installation.
4.3 The Alborz main tunnel excavation and support installation

The main tunnel has been excavated based on complementary site investigations that had been obtained in the service tunnel. Since cross section area of the main tunnel is bigger than the service tunnel and its cross section shape is horseshoe, so higher induced stress and concentration stress occur surrounding the main tunnel. The main tunnel has been excavated by conventional drilling & blasting and partial face heading and benching methods that firstly heading of the tunnel is excavated then benching will be excavated later. Therefore, heading excavation was modelled numerically as shown in Figure 12. After excavation of the main tunnel heading, support system with mechanical parameters like the Alborz service tunnel was installed.
The maximum displacement due to excavation of the main tunnel heading is about 11.5 cm. Figure 13 shows the plastic zone formed surrounding the Alborz main tunnel due to excavation with support installation.

![FLAC3D 5.01](image)

**Figure 13** The plastic zone formed surrounding the Alborz main tunnel.

In comparison with the service tunnel, the plastic zone of the main tunnel is wider than the service tunnel especially in the main tunnel floor. Therefore, support system in the main tunnel should be improved in respect to the Alborz service tunnel.

### 5 Capacity diagrams

In previous section, a numerical analysis was done assuming that the total bending moment $M$, thrust $N$ and shear force $Q$ acting on the equivalent section of thickness $t_{eq}$ and Young’s modulus $E_{eq}$. Figure 14 shows thrusts of elements with angles in respect to the horizon in a tunnel section of the Alborz service tunnel.

![FLAC3D 5.01](image)

**Figure 14** Thrust distribution in a section of the Alborz service tunnel.

The bending moments of elements with angles in respect to the horizon in a tunnel section of the Alborz service tunnel were determined using the numerical simulation done with FLAC$^3D$ as shown in Figure 15.
Once the values of bending moment, thrust and shear force on the equivalent section have been determined, the total bending moment $M$, thrust $N$ and shear force $Q$ can be distributed between each component of the sections (shotcrete & steel arch) based on equivalent section theory. In order to verify that the induced loading does not exceed admissible limits, the capacity diagrams of liners were constructed assuming $FS=0.67, 1, 1.5$.

Figure 16 represents the thrust-bending moment interaction diagrams for shotcrete and steel arches respectively computed based on the capacity diagram approach.

Figure 17 represents the thrust-shear force interaction diagrams for shotcrete and steel arches respectively computed based on the capacity diagram approach.

According to capacity diagrams in Figs. 16 &17, the composite liner installed in the Alborz service tunnel can resist against induced ground loading due to high overburden and squeezing condition.
6. Conclusion

A three dimensional numerical model was constructed for the service and one main Alborz tunnel with Flac. The equivalent section theory was used to calculate the composite liner (shotcrete & steel arches) equivalent parameters. After the numerical simulation of the Alborz service tunnel excavation and support installation, the resulting ground forces and bending moments applied on the composite liner were obtained. These resulting ground forces and bending moments were divided between shotcrete and steel arches according to the equivalent section theory. In order to verify that the induced loading does not exceed admissible limits, the capacity diagrams of each component of the liner were constructed. Based on these results, the support of the Alborz service tunnel is so sufficient that can resist against ground squeezing loading. However, the support of the Alborz main tunnel should be improved.

References


Tunnelling and Reinforcement in Heterogeneous Ground – A Case Study

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Abstract

A case study of tunnelling in heterogeneous ground conditions has been analysed. The case involves a tunnel excavated in mixed-face conditions, where the main host material was rock, but for a distance of about 30 m, the tunnel had to be driven through a thick layer of soil, primarily moraine and sandy soil materials. During tunnel drifting, a “chimney” cave developed through the soil layer, resulting in a surface sinkhole. This case was analysed using a three-dimensional numerical model with the FLAC3D software code, in which the soil stratigraphy and tunnel advance were modelled in detail. Tunnel and soil reinforcement in the form of jet grouting of the soil, pipe umbrella arch system, bolting, and shotcreting, was explicitly simulated in the model. The study aimed at comparing model results with observations and measurements of ground behaviour, and to replicate the major deformation pattern observed. The modelling work was based on a previous generic study in which various factors influencing tunnel and ground surface deformations were analysed for different cases of heterogeneous ground conditions. Model calibration was performed through adjusting the soil shear strength. The calibration provided a qualitatively good agreement with observed behaviour. Calculated deformations on the ground surface were in line with measured deformations, and the location of the tunnel collapse predicted by the model. The installed tunnel reinforcement proved to be critical to match with observed behaviour. Without installed pipe umbrella arch system, calculated deformations were overestimated, and exclusion of jet grouting caused collapse of the tunnel. These findings prove that, in particular, jet grouting of the soil layer was necessary for the successful tunnel advance through the soil layer.

1 Introduction

Tunnelling in urban areas has become increasingly challenging because of city development. Urban tunnelling usually implies that neither an optimal tunnel orientation nor an optimal tunnel depth can be achieved due to existing underground facilities or particular design needs (e.g., construction of a new station). A difficulty that often arises in urban areas is that the tunnel has to be excavated through mixed-face conditions with both soil and rock, which can have a major effect on the construction techniques and on the surroundings (see e.g., Clough & Leca 1993; U.S. Department of Transportation 2009; CEDD 2012).

This paper describes a case study of tunnelling in mixed ground conditions in the Stockholm area, and the post-construction numerical analyses. The objective of the study was to increase the understanding and knowledge of ground behaviour around tunnels in mixed ground conditions for
future application and implementation. Moreover, an assessment of the performance of the reinforcement measures used in the project, such as compensation grouting, umbrella arch system, shotcrete lining and bolts, was carried out via numerical analysis.

2 Stockholm City-Line: Passage under the Maria Magdalena Church

The Stockholm City Line is a 6 km long commuter train tunnel running between Tomteboda and Stockholm South. One part of this project involved tunnelling under the Maria Magdalena church in the southern part of Stockholm, and relatively close to the Stockholm South Station. The passage under the church was technically difficult due to mixed-face conditions, in addition to the thick layer of soil up to the ground surface. The tunnel train track is depicted in Figure 1, where the railway track is marked in red and the area of study is indicated in yellow.

![Figure 1](image.png)

Figure 1  Overview of the Stockholm City-Line tunnel track in the southern end, where the area of interest is surrounded in yellow, with the Maria Magdalena church next to it (Google Earth 2015)

During tunnel drifting, a "chimney" cave developed through the soil layer, resulting in a surface sinkhole with an area of 2 x 2 meters and 1.5 meters depth. This failure mechanism typical of frictional soils — flowing ground — is largely a construction design issue that should be avoided to the extent possible. Support measures like compensation grouting or pipe umbrella systems are well fitted to solve this kind of problems, and were studied in detail in this work to quantify their influence on the stability of the working area.

2.1 Geology

The geology consists of a thick layer of soil overlaying the bedrock. The dominating lithology of the area is "Stockholm granite", which was grouped into different rock types based on extensive rock mass characterisation as part of the City Line design work. The three rock types present in the area of interest were, according the nomenclature adopted for the design of the City Line tunnels: Rock type A ($RMR \geq 70$), Rock type B ($50 \geq RMR > 70$) and Rock type C ($30 \geq RMR > 50$).
The soil layer, in turn, comprised three different types with variable characteristics. The three soil types that were found through geotechnical pre-investigations were: sand (in the upper part, close to the ground surface), esker material (directly under the sand layer) and moraine (located between the layer of esker material and the rock). For practical reasons, only two soil types were included in the numerical model, where esker material and sand were forming one group and the moraine comprised the other type of soil.

Figure 2 illustrates the location of the tunnel with respect to the rock-soil interface, the approximate topography of the area, the situation of the moraine layer and the distribution of the rock types along the model.

![Figure 2](image)

**Figure 2**  Rock-soil interface (in grey), tunnel contour location (red), topography (brown) and moraine layer (magenta) in the area of the tunnel passage under the Maria Magdalena church from km 35+600 to km 35+700

### 2.2 Tunnelling procedure

This study focuses on a passage with a total length of 30 meters with mixed face conditions, excavated by the “drill-and-blast” method. The target of the study was the passage located between the km 35+633 and 35+663, for which the excavation was performed in three different phases of 10 meters each (phase A, phase B and phase C). A detailed description of the tunnelling procedure was given by Willer (2014), in which the collapse of the tunnel was also described. An explanatory scheme of the excavation sequence and reinforcements is shown in Figure 3.
The first step, and prior to the actual excavation, consisted of compensation grouting performed from the tunnel face. The tunnel was thereafter excavated with a split face and 1 meter round length, where the heading was first excavated for the total length of 30 meters, followed by the excavation of the bench.

The excavation and support sequence in phase A and phase B were as follows:

1. Symmetrical pipe umbrella arch, comprised by 60 pipes, each one of 15 meters length, with 7° outwards in the longitudinal axis and a covered angle of the cross section of 150°. The c/c distance was about 30 cm.
2. Conical excavation with round length of 1 meter, with 7° outward, with the maximal diameter at every 10 meter.
3. Systematic bolting, in the lower part of the arch, comprising 4 bolts (2 on each side) per excavated meter, with 5 meters length each and a c/c distance of 1 meter.
4. Shotcreting in the whole 180° arch, 35 cm thickness.

The excavation and support sequence in phase C differs from the previous ones and was as follows:

1. Asymmetrical pipe umbrella arch, comprised by 50 pipes, each one of 15 meters length, with 7° outward in the longitudinal axis and covering an angle of the cross section of 125°. The c/c distance of the pipes was about 30 cm.
2. Conical excavation with round length of 1 meter, with 7° outward, with the maximal diameter at the end of phase C.
3. Systematic bolting, in the lower part of the arch, comprising 5 bolts (2 on the left side and 3 on the right side) per excavated meter, with 5 meters length each and a c/c distance of 1 meter.
4. Shotcreting in the whole 180° arch, 35 cm thickness.
3 Model setup

The 3D numerical modelling code FLAC\textsuperscript{3D} (Itasca 2013) was used to model the ground surface settlements produced by tunnel excavation, and to assess the performance of the support systems that were used in this case study. It should be noted that the actual chimney cave and the soil flow was not simulated explicitly. The use of a continuum approach with FLAC\textsuperscript{3D} was considered appropriate given the characteristics of the rock mass and soil layers present in this study.

3.1 Geometry and boundary conditions

The model mesh was constructed based on the conical shape of the tunnel excavation. The separation layers for the different materials were created in the CAD program Rhinoceros (McNeel 2015), and imported to FLAC\textsuperscript{3D} as delimiting surfaces. The numerical model comprises 1,785,000 zones and was divided in regions with different zone sizes. The finest zone size elements are located adjacent to the excavation (0.25 m x 0.12 m x 0.20 m), with zones sizes increasing gradually towards the model boundaries. The geometry of the FLAC\textsuperscript{3D} model is shown in Figure 4.

Stresses were initialized in the model in each element. The boundary conditions applied to the model consisted of roller boundaries for the vertical sides of the model, pinned boundary condition for the bottom of the model, and a free surface for the ground surface.

![Figure 4](https://via.placeholder.com/150)  
*Figure 4* Finite difference mesh in the numerical model used to analyse the passage under the Maria Magdalena church, showing longitudinal and transversal sections in the middle of the model, including the excavation area
3.2 Initial stress

The initial stress conditions in the rock mass were taken from measurements performed for the Stockholm City-Line at Södermalm (described in Perman & Sjöberg 2007), and given as follows:

\[
\begin{align*}
\sigma_H &= 2.0 + 0.125z \\
\sigma_h &= 1.0 + 0.100z \\
\sigma_v &= 0.0265z
\end{align*}
\]

where:
- \(\sigma_H\) = maximum horizontal principal stress in MPa; orientation 160° from Geographic North,
- \(\sigma_h\) = minimum horizontal principal stress in MPa,
- \(\sigma_v\) = vertical stress in MPa,
- \(z\) = depth from ground surface in meters.

For the soil layer, the stress state was assumed to be lithostatic and given by:

\[
\sigma_H = \sigma_h = K_0 \sigma_v
\]

where:
- \(K_0\) = initial stress ratio (dimensionless)

3.3 Material properties

The rock and soil materials in the FLAC³D model were represented using a linear elastic-perfectly plastic Mohr-Coulomb constitutive model with tensile strength cut-off. The assumption that the compensation grouting would only affect the soil layers within a radial distance of 1 meter from the tunnel contour was made. The grouted soil material was thus simulated as a soil with increased cohesive and tensile strength, with properties representing a poor quality concrete (as this was the target strength for the conducted grouting). The material properties for the rock and soil layers (cf. Section 2.1) are shown in Table 1.
Table 1  Rock mass and soil properties used in FLAC$^{3D}$. (Handboken Bygg: Geoteknik 1984; Lindfors 2008, Larsson 2008; Larsson et al. 2007; PLAXIS Material models manual 2015, Trafikverket 2014)

<table>
<thead>
<tr>
<th></th>
<th>Density ρ (kg/m$^3$)</th>
<th>Young’s modulus $E_m$ (MPa)</th>
<th>Poisson’s ratio $v_m$ (-)</th>
<th>Cohesion $c$ (MPa)</th>
<th>Friction angle $\phi$ (°)</th>
<th>Tensile strength $\sigma_t$ (MPa)</th>
<th>Dilation angle $\psi$ (°)</th>
<th>$K_0$ (-)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rock type A</td>
<td>2650</td>
<td>69000</td>
<td>0.25</td>
<td>6.6</td>
<td>58.3</td>
<td>2.4</td>
<td>7</td>
<td>-</td>
</tr>
<tr>
<td>Rock type B</td>
<td>2650</td>
<td>46000</td>
<td>0.25</td>
<td>2.5</td>
<td>58.9</td>
<td>0.5</td>
<td>7</td>
<td>-</td>
</tr>
<tr>
<td>Rock type C</td>
<td>2650</td>
<td>11000</td>
<td>0.25</td>
<td>1.0</td>
<td>51.9</td>
<td>0.08</td>
<td>0</td>
<td>-</td>
</tr>
<tr>
<td>Sand</td>
<td>1600</td>
<td>20</td>
<td>0.35</td>
<td>0.0</td>
<td>35</td>
<td>0</td>
<td>5</td>
<td>0.43</td>
</tr>
<tr>
<td>Moraine</td>
<td>2000</td>
<td>100</td>
<td>0.35</td>
<td>0.0</td>
<td>45</td>
<td>0</td>
<td>15</td>
<td>0.30</td>
</tr>
<tr>
<td>Grouted soil</td>
<td>2000</td>
<td>200</td>
<td>0.25</td>
<td>0.1</td>
<td>35</td>
<td>0.2</td>
<td>5</td>
<td>0.43</td>
</tr>
</tbody>
</table>

3.4  Ground support properties

The shotcrete was represented as a liner element and the fully-grouted bolts were modelled as cable elements (only accounting for axial forces) in FLAC$^{3D}$. In order to consider the bending resistance provided by the pipe umbrella system, the pipes were represented using pile structural elements. It was assumed that only the pipes were filled with grout, not the area between the pipe and the soil (Volkmann & Schubert 2009). Figure 5 represents the structural elements that were included in the excavation sequence, with the material properties for the support elements shown in Table 2. The interaction parameters between the structural elements and the surrounding material are given in Table 3 through Table 5.

![Figure 5](image-url)  Structural elements in FLAC$^{3D}$ model after the completion of the 30 meter tunnel excavation
Table 2  Properties for rock bolts, pipe elements and shotcrete used in FLAC$^{3D}$. (Rosengren 2004; Malmgren 2005; Holmberg 2014)

<table>
<thead>
<tr>
<th></th>
<th>Density $\rho$ (kg/m$^3$)</th>
<th>Young's modulus $E_m$ (GPa)</th>
<th>Poisson's ratio $\nu$</th>
<th>Diameter $d$ (mm)</th>
<th>Thickness $t$ (mm)</th>
<th>Length $l$ (m)</th>
<th>Characteristic compressive strength $f_{ck}$ (MPa)</th>
<th>Characteristic tensile strength $f_y$ (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bolt</td>
<td>7800</td>
<td>200</td>
<td>0.3</td>
<td>20</td>
<td>-</td>
<td>5</td>
<td>246</td>
<td>246</td>
</tr>
<tr>
<td>Pipe element</td>
<td>7800</td>
<td>200</td>
<td>0.3</td>
<td>160</td>
<td>10</td>
<td>15</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Shotcrete</td>
<td>2300</td>
<td>16</td>
<td>0.25</td>
<td>-</td>
<td>350</td>
<td>-</td>
<td>12</td>
<td>3.9</td>
</tr>
</tbody>
</table>

Table 3  Properties for bolt-grout interface used in FLAC$^{3D}$. (Rosengren 2004; Itasca 2010)

<table>
<thead>
<tr>
<th></th>
<th>Shear modulus $G$ (GPa)</th>
<th>Stiffness $(GPa/m)$</th>
<th>Thickness $t$ (mm)</th>
<th>Compressive strength $\sigma_c$ (MPa)</th>
<th>Cohesion $c$ (KN/m)</th>
<th>Friction angle $\phi$ $(^\circ)$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bolt-grout interface</td>
<td>9.0</td>
<td>8.15</td>
<td>10</td>
<td>20.0</td>
<td>565</td>
<td>40</td>
</tr>
</tbody>
</table>

Table 4  Properties for pipe-soil interface used in FLAC$^{3D}$. (Itasca 2012)

<table>
<thead>
<tr>
<th></th>
<th>Shear stiffness $k_s$ (GPa)</th>
<th>Normal stiffness $k_n$ (GPa)</th>
<th>Cohesion $c$ (MPa)</th>
<th>Friction angle $\phi$ $(^\circ)$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pipe-soil interface</td>
<td>200</td>
<td>2000</td>
<td>0.0</td>
<td>35</td>
</tr>
</tbody>
</table>

Table 5  Properties for shotcrete-rock interface used in FLAC$^{3D}$. (Malmgren 2005; Itasca 2010)

<table>
<thead>
<tr>
<th></th>
<th>Shear stiffness $k_s$ (GPa/m)</th>
<th>Normal stiffness $k_n$ (GPa/m)</th>
<th>Adhesion (MPa)</th>
<th>Cohesion $c$ (MPa)</th>
<th>Friction angle $\phi$ $(^\circ)$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Shotcrete-rock interface</td>
<td>0.1</td>
<td>1.0</td>
<td>0.6</td>
<td>0.5</td>
<td>40</td>
</tr>
</tbody>
</table>

4  Model calibration

The calibration of a numerical model against measurements and observations is a key aspect to ensure that results are representative and realistic. Therefore, the model was first calibrated using monitoring data from ground surface settlements at 8 different points, obtained from levelling pins located at the ground surface, more or less in the centre of the cross-section of the tunnel, as shown in Figure 6.

Based on the associated generic study on tunnelling in heterogeneous ground, as part of this overall project (Eriksson et al. 2016), the following assumptions were made: (1) the initial stress state in the rock mass would not influence the main behaviour of the model for mixed ground conditions, (2) experiences from the construction site showed that the soil layer was above the groundwater level and thus unsaturated (Stille 2015), (3) the rock would not be affected significantly by the pore pressures, and (4) the soil properties would have a major influence on the deformations when tunnelling. Therefore, the calibration of the model was done by varying the cohesion of the frictional soil and the moraine.

For calibration purposes, the cohesion of the soil layers in the model was varied between 0 and 5 kPa, showing large sensitivity to these values. The best-fit model achieved for 2 kPa cohesion (checked against measured settlements, see Figure 7) was used for investigating two extra cases,
aiming to check the effectiveness of the umbrella arch system and the compensation grouting. The "chimney" cave that developed through the soil layer when drifting was not simulated explicitly, however, the model showed that the maximum displacements occurred in km 35 + 649, coincident with the location where the cave started to develop.

Figure 6  Overview of the Maria Magdalena church area, where the zone with available ground surface measured displacements marked in red and the location of measurement pin marked in black (modified after Trafikverket 2015)

Figure 7  Calculated ground surface settlements for the 8 calibration points for the calibration models, together with the actual surface settlements
5 Evaluation of support measures

The ground surface settlements for the best-fit model, with and without umbrella arch system, are shown in Figure 8, together with measured settlements. These results indicate that much larger surface settlements develop if the umbrella arch system is not included in the model. The simulated results regarding the best-fit model without compensation grouting showed that large-scale collapse occurred in the model, starting from the vicinity of the tunnel and propagating upwards up to the ground surface.

![Figure 8](image.jpg)

Figure 8 Calculated ground surface settlements for the 8 calibration points for the best-fit model with and without umbrella arch system, together with the actual surface settlement results

A comparison between the best-fit models with and without umbrella arch system was done with focus on ground support elements, in order to prove its effectiveness. This evaluation zeroed in on displacements and acting moments in the liner on the one hand, and forces developed in the bolts, on the other hand.

The influence of the umbrella arch system on liner displacements is obvious (see Figure 9), where this type of support helps to minimize the liner deformations, mainly in the crown of the tunnel. Additionally, the acting moments in the liner are somewhat redistributed (see Figure 10), indicating that the pile elements in the FLAC\(^3D\) model (umbrella arch system) contribute in such a way that the moment acting along the longitudinal axis of the liner is reduced. The influence of the umbrella arch system on the bolt elements is however minor (see Figure 11), since the cable elements used in the FLAC\(^3D\) model only account for axial forces.

6 Discussion

Based on the results obtained from the performed analyses, the calibration of the model against monitored surface settlements was in good agreement with the actual behaviour. Furthermore, the location of the collapse that occurred in the tunnel was captured by the numerical model as well. The results further showed that the ground surface settlements were extremely sensitive to variations of...
the shear strength of the soil layers, especially for the cases with low cohesion, where a change from 2.0 to 0.0 kPa lead to increased ground surface settlements of up to 100%. The ground support measures also influenced the results significantly. On the one hand, the model without the umbrella arch system yielded up to 140% larger surface settlements compared to the case with the umbrella arch included. On the other hand, the model without compensation grouting resulted in large-scale collapse of the model.

![Figure 9](image)

**Figure 9** Calculated liner displacements for the numerical models after the excavation of the passage, where the values are expressed in meters.

![Figure 10](image)

**Figure 10** Calculated liner acting moments along the longitudinal axis for the numerical models after the excavation of the passage, where the values are expressed in Nm.
Figure 11  Calculated axial forces acting on bolts for the numerical models after the excavation of the passage, where the values are expressed in N

It should be pointed out that the assumption of the jet grouting acting as an element that mainly increases the cohesion of the soil layer surrounding the tunnel was made. However, it is possible (or perhaps even likely) that some areas or volumes do not have enough grouted soil, and therefore have not acquired the properties that were expected in the design stage.

The work that has been presented in this paper can be used as a guide for required pre-investigations in feasibility studies and early design stages, as well as a basis for a numerical modelling methodology. In cases where mixed ground conditions are present, the soil characteristics "dominate" the tunnel behaviour. Thus, a detailed characterisation of rock properties and rock stresses are not necessary. Instead, efforts should be focused on: (i) determining the extent and location of soil layers, and (ii) determining the strength and stiffness properties of the soils. While determining the soil (and rock) stratigraphy is often part of pre-investigations, soil characteristics are often assumed or estimated conservatively. A more detailed investigation of soil properties, including e.g. triaxial tests, and for relevant stress conditions is recommended for this types of cases. A sensitivity analysis of the soil properties can be also beneficial, since these properties are critical with respect to the general behaviour on the model.

7 Conclusions

The use of a three-dimensional model, with a realistic representation of both the actual geological conditions and the excavation sequence, allowed obtaining a good match between model results and monitored deformations on the ground surface. The FLAC$^{3D}$ model results also proved the effectiveness of both the pipe umbrella arch system and compensation grouting in preventing collapse of the tunnel during drifting, where the latter showed to have an even more critical contribution to the stability than the pipe umbrella arch system. Apart from this, the pipe umbrella arch system provided a large reduction of the surface settlements.
A three-dimensional numerical model is considered to be an adequate tool to obtain reliable results and good understanding regarding deformation patterns, general tunnel stability and performance of the ground support measures. Furthermore, this type of model can be used to investigate different design solutions and check their influence on neighbouring structures in an urban scenario.

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