in Mining & Underground Construction

E. Villaescusa & Y. Potvin

- editors -

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GROUND SUPPORT IN MINING AND UNDERGROUND CONSTRUCTION

PROCEEDINGS OF THE FIFTH INTERNATIONAL SYMPOSIUM ON GROUND SUPPORT, 28–30 SEPTEMBER 2004, PERTH, WESTERN AUSTRALIA

Ground Support in Mining and Underground Construction

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Cover: Ground support in development heading following multiple rockbursts (Provided by Professor E.Villaescusa)

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Foreword

Ground Support in Mining and Underground Construction—Villaescusa & Potvin (eds.) © 2004 Taylor & Francis Group, London, ISBN 90 5809 640 8

The Fifth International Symposium on Ground Support in Mining and Underground Construction was held by the Australian Centre for Geomechanics and the Western Australian School of Mines, at Perth, Australia from September 28 to 30, 2004. The Symposium follows on from international symposia held at Lulea, Sweden, 1983, Sudbury Canada, 1992, Lillehamer, Norway, 1997 and Kalgoorlie, Australia, 1999. The objective of the Symposium was to exchange experiences, knowledge and lessons learnt in ground support with special attention being given to mining applications and underground construction.

The Symposium dealt with twelve main themes:

- 1. Case studies;
- 2. Rock mass characterization;
- 3. Modelling;
- 4. In situ and laboratory testing;
- 5. Open pit;
- 6. Dynamic testing;
- 7. Rockfalls and failure and mechanisms;
- 8. Civil engineering and tunnelling;
- 9. Design;
- 10. Corrosion;
- 11. Surface support;
- 12. Other support.

A total of sixty one papers have been published in these proceedings. In addition, two Keynote Addresses were also published.

Keynote Lectures

E.T.Brown, Australia: The dynamic environment of ground support and reinforcement;

C.R.Windsor, Australia: A review of long, high capacity reinforcing elements in rock engineering practice.

The organizing committee wishes to thank all the supporting organizations and the authors for their valuable contributions. Ground support remains essential to sustain and progress prosperous mining and civil engineering industries.

E.Villaescusa Y.Potvin

Organization

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Keynote lectures

The dynamic environment of ground support and reinforcement

E.T.Brown

Golder Associates Pty Ltd, Brisbane, Queensland, Australia

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ABSTRACT: This paper is intended to act as an introduction to the symposium by providing an overview of the state-of-the-art of ground support and reinforcement and, in particular, of the advances made in the five years since the last symposium in this series. Support and reinforcement elements and systems available for application in both static and dynamic loading conditions are considered. The available methods of analysis and modelling are reviewed. Finally, the overall performance achieved by support and reinforcement systems is considered, particularly from the perspective of the reduction of injuries and fatalities from rockfalls. Throughout, emphases are placed fundamental principles and on underground metalliferous mining.

1 INTRODUCTION

This symposium is the fifth in a series of international symposia which began at Abisko, Sweden, in 1983 (Stephansson 1984). The most recent symposium in the series was held at Kalgoorlie, Western Australia in 1999 (Villaescusa et al. 1999). At that symposium, the author presented a keynote paper that sought to provide a summary account of the evolution of support and reinforcement philosophy and practice in underground mining (Brown 1999a). On this occasion, the opportunity will be taken to review the state-of-theart of ground support and reinforcement in underground excavations in rock and, in particular, the advances made in the five years since the time of the last symposium in this series. Because of the symposium's location in Western Australia and the author's recent professional interests, the emphasis will again be placed on hard rock mining applications, although not to the total exclusion of underground coal mining and civil construction. A significant development in Western Australia, and elsewhere, in the last five years has been the increased emphasis placed on the dynamic capabilities of support and reinforcement systems. Accordingly, particular attention will be given to dynamic capable systems. The techniques and systems used for conventional static or pseudostatic loading will be considered in section 2 and those for dynamic loading in section 3.

Because of its inherent logic and the fact that it finds widespread use, particularly in the Australian mining industry, the distinction between support and reinforcement due to Windsor and Thompson (1993) will be made here.

2 STATIC AND PSEUDO-STATIC SUPPORT AND REINFORCEMENT SYSTEMS

2.1 Rock and cable bolts

It is perhaps remarkable to find that, although rock and cable bolts have been used in underground mining and construction for several decades (if not more than 100 years in the case of rock bolts), bolt elements and bolting systems continue to evolve and improve. The papers presented to this symposium detail advances made in fully encapsulated resin and cement grouted bolts (Mikula 2004, Mould et al. 2004, Neindorf 2004), one pass mechanized bolting (Mikula 2004, Neindorf 2004) and bulbed cables (Yumlu & Bawden 2004), for example. The developments in ground support practices that have accompanied greater productivity, larger excavations and larger equipment are especially well-illustrated in the paper by Neindorf (2004) describing the evolution of ground support practices at the Mount Isa mine over the past 30 years.

In a detailed and valuable review paper, Windsor (2004) concludes that "the quality and performance of cable bolts used to stabilise temporary, non-entry, production excavations have improved over the last 20 years to the point where they are now an essential part of modern mining practice. Cable bolts have provided the industry with increased production, increased safety and increased flexibility in the extraction process. However, with the development of wider span haulage and other larger mine openings, cable bolts are now also used to secure longer life, infrastructure excavations." Windsor (2004) recommends "that greater care and attention to detail be invested during selection and installation of cable bolts for mine infrastructure excavations than that given to mine production excavations". He identifies, in particular, the importance of the control of the geometry, material quality, installation and testing of the barrel and wedge fittings used as cable grips.

It is also important to recognize that the use and effectiveness of rock and cable bolts in Australia's underground coal mines have developed considerably in the recent past. Hebblewhite et al. (2004) suggest that the significant trends over the last decade have included:

- use of longer bolts;
- use of partial and predominantly full-encapsulation, polyester resin anchored bolts;
- use of threaded bolt fixing systems;
- adoption of bolt pre-tensioning in an increasing number of applications;
- · adoption of different grades of steel to achieve stiffer and stronger bolts; and
- variations to bolt deform patterns and ribbing systems for improved anchorage and load transfer performance.

An issue that has long existed, but has often been over-looked, is the corrosion resistance and longevity of rock and cable bolts. The initial Snowy Mountains installations which are generally regarded as having pioneered the systematic use of rock bolting in Australia (e.g. Brown 1999b) are now more than 50 years old. It was inevitable, therefore, that this issue would assume the increasing importance accorded it by the papers presented to this symposium (e.g. Bertuzzi 2004, Hassell et al. 2004, Hebblewhite et al. 2004, Satola & Aromaa 2004, Windsor 2004). As noted by Hassell et al. (2004) and Potvin & Nedin (2004), the long-term corrosion resistance of the popular friction rock stabilizers, remains an issue. Corrosion protection is one of the advantages offered by fully encapsulated bolts and cables. However, there are suggestions that cement grouting alone does not provide long-term (e.g. 100 year) corrosion protection (Bertuzzi 2004). For long-term protection, two independent corrosion barriers are usually required. Depending on the atmosphere and the mineralogy and groundwater conditions in the rock mass, corrosion may also affect surface fixtures such as plates and nuts as well as the bolts and cables themselves. Of course, galvanizing provides protection to the steel underneath but not necessarily for long periods of time (Hassell et al. 2004, Windsor 2004).

Interestingly, in a detailed inspection of 50 km of 35–40 year old tunnels in the Snowy Mountains Scheme, Rosin & Sundaram (2003) found the mainly fully cement grouted, hollow core mild steel bolts to be in excellent condition, showing little evidence of corrosion. An approximately 5 mm protective grout or bitumen coating applied to the bolt threads and face plates appeared to have worked very well. Carefully controlled installation and grouting is a necessary pre-condition for the achievement of such performance (Windsor 2004).

With increasing knowledge, experience and the availability of a range of analytical and numerical tools, rock and cable bolt installations are now being designed for increasingly demanding operational conditions in both civil engineering and underground mining. However, the most successful installations are usually those whose performance is monitored by a well-designed instrumentation system as part of a systematic observational approach (e.g. Moosavi et al. 2004, Thibodeau 2004, Thin et al. 2004, Tyler & Werner 2004, Yumlu & Bawden, 2004).

2.2 Shotcrete

Over the last decade, increasing use has been made of shotcrete for ground support and control in infrastructure, development and production excavations in underground mines in Australia and elsewhere. Clements (2003) reports that nearly 100,000 m³ of shotcrete is applied annually in some 20 underground mines in Australia. Advances have been made in mix design, testing, spraying technology and admixtures which have combined to improve the effectiveness of shotcrete. Wet-mix fibre-reinforced shotcrete is now the industry standard.

Of course, shotcrete has long been an essential part of support and reinforcement systems in underground civil construction where its use is well-established even for softer ground than that commonly met in underground mining (Kovari 2001). In underground mining, shotcrete is now used to good effect not only for infrastructure excavations, in weak ground (e.g. Yumlu & Bawden, 2004), for rehabilitation, and in heavy static or pseudo-static loading conditions (e.g. Tyler & Werner 2004), but as a component of support and reinforcement systems for dynamic or rockburst conditions (e.g. Li et al. 2003, 2004). The toughness or energy absorbing capacity of fibre-reinforced shotcrete is

particularly important in this application. A new toughness standard, the Round Determinate Panel test, has been developed in Australia and adopted in some other countries (Bernard 2000, 2003). The performance of fibre-reinforced shotcrete measured in these tests can vary significantly with the type (usually steel or polypropylene structural synthetic fibres) and dosage of fibres used.

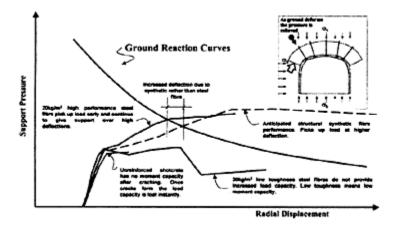


Figure 1. Ground-support interaction diagram illustrating the effects of fibre type and dosage on the strength and ductility developed by fibre-reinforced shotcrete (Papworth 2002).

Figure 1 uses a ground-support interaction diagram to provide a conceptual illustration of some of the effects of fibre type and dosage on the strength and ductility developed in fibre reinforced shotcrete (Papworth 2002).

2.3 Mesh and sprayed liners

Another important change in support and reinforcement practice in underground mining in recent years has been the increasing emphasis being placed on mesh and sprayed liners of several types as a primary ground control mechanism. Although, because of the large quantities used and its importance as a support technique, shotcrete has been treated here as a special category of support, it is often included with other techniques in the class of spray-on liners (e.g. Spearing & Hague 2003). The overall subject of mesh and sprayed liners has become so significant that it now has its own series of specialist international meetings.

In some mining districts such as those in Western Australia and Ontario, Canada, mining regulations and codes of practice now require that some form of surface support, usually mesh, be used in all personnel entry excavations. In Western Australia, the Code of Practice applies to all headings that are higher than 3.5 m and requires that surface support be installed down to at least 3.5 m from the floor (Mines Occupational Safety and

Health Advisory Board 1999). These provisions form part of the steps being taken to understand and alleviate the rockfall hazard in Western Australia's, and Australia's, underground metalliferous mines (Lang & Stubley 2004, Potvin & Nedin 2004).

The most commonly used mesh is probably welded mesh made of approximately 5 mm thick steel wire and having 100 mm square openings. The steel wire may be galvanised or not. The alternative has been an interwoven mesh known as chain link mesh. The disadvantage of traditional chain link mesh compared with weld mesh has been the difficulty of applying shotcrete successfully through the smaller openings available. This difficulty has now been overcome in a high strength, light weight chain link mesh with 100 mm openings which is easy to handle and can be made to conform to uneven rock surfaces more readily than weld mesh. A feature of this mesh is the fact that the intersections of the wires making up the squares in the mesh are twisted rather than simply linked or welded. Roth et al. (2004) describe static and dynamic tests on this mesh. Mesh of this type is being used successfully at the Neves Corvo Mine, Portugal, where it has been particularly successful in rehabilitating damaged excavations. Li et al. (2004) report that this mesh is being trialled by St Ives Gold, Western Australia. Tyler & Werner (2004) refer to recent trials in sublevel cross-cuts at the Perseverence Mine, Western Australia, using what a similar Australian made high strength chain link mesh. It is understood that completely satisfactory mechanised installation methods have yet to be developed.

In this symposium, Hadjigeorgiou et al. (2004) and Van Heerden (2004) discuss the use of cementitious liners to support, protect and improve the operational performance of ore passes in metalliferous mines. One of the benefits of cementitious liners is the corrosion protection that they provide to the reinforcing elements. Both papers emphasise the need to consider the support and reinforcement of ore passes on a cost-effectiveness basis taking into account the need to rehabilitate or replace failed passes. The author has had the experience of having to recommend the filling with concrete and re-boring of critical ore passes that had collapsed over parts of their lengths.

Although their use was referred to at the 1999 symposium, there have been significant developments in the use of thin, non-cementitous, spray-on liners (TSLs) since that time (e.g. Spearing & Hague 2003). These polymer-based products are applied in layers of typically 6 mm or less in thickness, largely as a replacement for mesh or shotcrete. Stacey & Yu (2004) explore the rock support mechanisms provided by sprayed liners. The author's experience at the Neves Corvo Mine, Portugal, is that TSLs are useful in providing immediate support to prevent rock mass deterioration and unravelling in special circumstances (Figure 2), but that they do not yet provide a cost-effective replacement for shotcrete in most mainstream support applications. In some circumstances, they can be applied more quickly than shotcrete and may be used to provide effective immediate support when a fast rate of advance is required. Recently, Archibald & Katsabanis (2004) have reported the effectiveness of TSLs under simulated rockburst conditions.



Figure 2. Localised application of a thin, spray-on liner in a drift at the Neves Corvo Mine, Portugal.

2.4 Support and reinforcement in the mining cycle

Overcoming the limitations and costs associated with the cyclic nature of underground metalliferous mining operations has long been one of the dreams of miners. More closely continuous mining can be achieved in civil engineering tunnelling and in longwall coal mining than in underground hard rock mining. Current development of more continuous underground metalliferous mining systems is associated mainly, but not only, with caving and other mass mining methods (Brown 2004, Paraszczak & Planeta 2004).

Several papers to this symposium describe developments that, while not obviating the need for cyclic drill-blast-scale-support-load operations, will improve the ability to scale and provide immediate support and reinforcement to the newly blasted rock. Jenkins et al. (2004) describe mine-wide trials with hydro-scaling and in-cycle shotcreting to replace conventional jumbo scaling, meshing and bolting at Agnew Gold Mining Company's Waroonga mine, Western Australia. Neindorf (2004) also refers to the possibility of combining hydro-scaling with shotcreting to develop a new approach to continuous ground support in the development cycle at Mount Isa. These developments form part of the continuous improvement evident in support and reinforcement practice in underground mining.

2.5 Backfill

As was noted at the 1999 symposium, although backfill has been used to control displacements around and above underground mining excavations for more than 100

years, the great impetus for the development of fill technology came with the emergence of the "cut-and-fill era" in the 1950s and 60s (Brown 1999a). It was also noted that fill did not figure prominently in the papers presented to that symposium. A few years earlier, paste fill made from mill tailings and cement and/or other binders, had been developed in Canada (Landriault 2001). Since that time, the use and understanding of paste fill have increased dramatically, so much so that Belem et al. (2004b) suggest that it is "becoming standard practice in the mining industry throughout the world".

Cemented paste fill is now used with a range of mining methods including sublevel open stoping, cut-and-fill and bench-and-fill. In some applications, it is necessary that unsupported vertical paste fill walls of primary stopes remain stable while secondary stoping is completed. In common with Landriault (2001) and Belem et al. (2004a), the author has had success using the design method proposed by Mitchell (1983). A particular requirement in some applications is to include enough cement to prevent liquefaction of the paste after placement (Been et al. 2002).

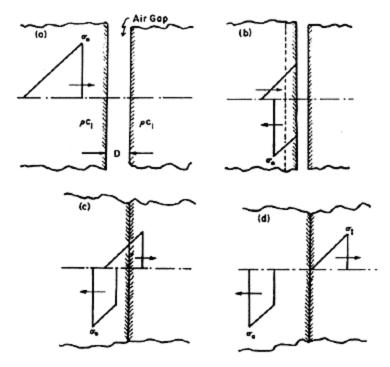
In two papers to this symposium, Belem et al. (2004a, b) discuss a range of fundamental and applied aspects of the use of cemented paste fill in cut-and-fill mining generally, and in longhole open stoping at La Mine Doyen, Canada. Varden & Henderson (2004) discuss the use of the more traditional cemented rock fill to fill old underground mining voids at the Sons of Gwalia Mine, Western Australia.

3 DYNAMIC SUPPORT AND REINFORCEMENT SYSTEMS

3.1 Fundamental considerations

Several of the world's mining districts are having to deal increasingly with mininginduced seismicity and the related rockbursts. The increasing incidence of mine seismicity and rockbursts is generally associated with increasing depths of mining but it may also be influenced by other factors, such as the high horizontal stress regime encountered in Western Australia. In the 1999 symposium, only five papers, all of them from South Africa, dealt specifically with support and reinforcement in burst-prone ground. By the author's preliminary count, at least 10 papers in the 2004 symposium, only one of which is from South Africa, are concerned with support and reinforcement under dynamic loading conditions.

It is widely accepted that there are two modes of rock mass response that lead to instability, mine seismicity and rockbursts—slip on natural or mining-induced planes of weakness, and fracture of the intact rock itself, usually close to excavation boundaries (Brady & Brown 2004). In either case, excess energy will be released from around the source of the instability and propagate through the rock mass as a series of seismic waves.



STRESS TRANSIENTS IN SOLIDS

Figure 3. Mechanics of closure and the filtering action of an air gap acted on by a stress transient: (a) approach of the transient; (b) reflection and beginning of closure; (c) shortly after closure; and (d) distribution of stress at the instant interaction with the joint is complete (Rinehart 1975).

These waves will induce dynamic stresses and associated displacements within the rock mass. As well as compression and shear body waves, surface waves may result near excavation boundaries. Waves may be refracted and reflected at interfaces and boundaries of various kinds (Rinehart 1975). Figure 3 shows the simple example of the closure and filtering action of an air gap, D, acted on by a triangular stress transient of peak magnitude σ_0 .

In this, as in other branches of engineering, attention must be paid to terminology which is sometimes used loosely or even incorrectly. We are concerned here with **dynamic loading** which, in general engineering terms, varies with time and may arise

from repeated loads, moving loads, impact loads, shock waves or seismic waves. Dynamics concerns the motion of bodies as well as the forces and stresses applied to them. **Impact loading** is a particular form of dynamic loading that is applied suddenly when two bodies collide. The inertia of the body being impacted has an important influence on the mechanical effects of impact loading. **Static loading**, on the other hand, arises from forces that are applied slowly and then remain nearly constant with time (Tamboli et al. 2004). The term **pseudo-static loading** is used to describe loads that, while not truly static in the sense of this definition, may be treated as static in terms of the stresses and deformations induced in the loaded body.

The essential differences between static or pseudo-static loading and the dynamic loading experienced during seismic events leading to rockbursts are that, in the latter case:

- the support and reinforcing elements and systems may be subjected to impact or impulsive loading that imposes maximum loads and deformations that are well in excess of those experienced in the comparable static case;
- the energy, or part of the energy, released by the seismic event will have to be absorbed somewhere in the rock-support-reinforcement system; and
- the requirement for the containment of disturbed and broken rock around the excavation periphery will be greater.

It must also be remembered that engineering materials have different strength and stiffness properties under dynamic than under static loading (Tamboli et al. 2004).

As Li et al. (2003, 2004) note, the most commonly used approach to the design of dynamically capable support and reinforcement systems for underground rockburst conditions is based on energy considerations. Rojas et al. (2004) provide an example of the use of the energy approach in the design of support for rockburst conditions. In the energy approach, it is postulated that the damaged rock mass around an excavation releases a certain amount of energy and that the support and reinforcement system must be capable of absorbing this energy. This usually requires that the reinforcement elements should possess yielding capability for a specified velocity and displacement. This has led to an emphasis being placed on the development of yielding reinforcing elements. As Li et al. (2003, 2004) have pointed out, some, and often all, of the assumptions and requirements of this simple approach may not be satisfied in practice.

The dynamic loading of the rock mass and support system (for convenience in this discussion taken to mean the support and reinforcement system) in a seismic or rockburst event is a very complex process. From a mechanistic perspective, there is an initial acceleration of the rock mass induced by the stress waves. This will impose dynamic loading on the surface support elements and fixtures as well as on the reinforcing elements. At some point, the accelerated rock mass and support system will reach their maximum velocities which may, or may not, be the same for the rock mass and the support system elements. To mobilise the full support system capacity and to maintain the integrity of the rock mass-support system, the rock mass and support system must decelerate from the peak ejection velocity over a short period of time.

The ability of the surface support to accommodate these sudden changes in velocity is of vital importance to the effective dynamic performance of the system. Li et al. (2003, 2004) suggest that momentum change theory can be useful in establishing the

requirements in this regard. The fundamental importance of momentum in the analysis of stress transients in solids has been pointed out by Rinehart (1975) who observed that "an impulsively applied blow introduces momentum into the system to which it is applied. Momentum is similar to energy in that it cannot be destroyed but it has the added advantageous quality that it cannot change its identity and can be kept track of easily. It always appears as mechanical motion which moves about through a system distributing itself in various ways."

Relating the momentum change to the resisting force, F, applied over a period of time produces the well-known equation F=ma where m is the mass of the system and a is the acceleration (or deceleration) to which it is subjected (Li et al. 2003, 2004). A typical representation of dynamic loading used in earthquake and civil engineering uses waveform characteristics as input and gives forces, displacements and displacement rates at output. However, exact dynamic analysis is usually only possible for simple structural systems (Tamboli et al. 2004). Nevertheless, when momentum change is considered in the design of a dynamically capable support system, it introduces an important second criterion to be satisfied in addition to the energy absorption criterion. Rinehart (1975) presents solutions to a number of idealised problems involving surfaces and interfaces that are instructive in the present context. In order to develop a more complete method of analysis for dynamically loaded rock-support systems around underground excavations, more research such as that reported by Cichowicz et al. (2000), Milev et al. (2003) and Simser & Falmagne (2004) is required into the seismic source parameters and waveforms of mining-induced seismic events.

3.2 Dynamic capable support and reinforcement elements and systems

Several papers presented to the symposium report details of dynamically capable support and reinforcing elements and systems and of their performance under test and service conditions. Player (2004) discusses the introduction of cone bolts at the Big Bell Mine, Western Australia, in 1999 and subsequent experience with testing, installation, stress corrosion and performance of the cone bolts in increasingly demanding applications. Falmage and Simser (2004) outline Canadian experience with rockburst support systems and the development of the resin grouted Modified Cone Bolt (MCB) and the Rockburst Support System using MCBs and de-bonded yielding cables introduced at the Brunswick Mine, Canada, in 2001. Gaudreau (2004) also describes the use of the MCB and a yielding cable bolt as part of the support and reinforcement used under what are classified as conventional rockburst, full rockburst and deep squeezing conditions at the Brunswick Mine. Gaudreau et al. (2004) provide details of the testing systems and analytical methods used to assess the performance of tendons under dynamic loading.

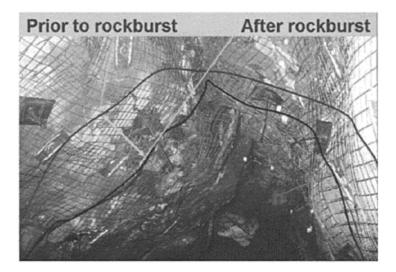


Figure 4. Drive profiles before and after a large rockburst in which the broken rock mass was contained by the dynamic support system (Li et al. 2004).

Li et al. (2004) describe the development of a yielding cable with a sliding anchor and an energy absorbing plate made from conveyor belt rubber and their application as part of dynamically capable support and reinforcement systems by the St Ives Gold Mining Company, Western Australia. Figure 4 shows the profiles of a drive supported with this system before and after a rockburst of approximately 1.5–2.0 local magnitude. Convergence of the drive was up to 0.7 m over a 20 m length but the fragmented rock around the excavation was fully contained.

3.3 Testing systems

The design of rock and cable bolt testing systems to replicate the loading conditions occurring in practice, particularly the dynamic loading resulting from rock-bursts, is extremely challenging. In a review of known systems carried out in 2002, the author found that, although particular elements of the total rock mass-support-reinforcement system and its loading may be represented satisfactorily, it is extremely difficult to replicate complete seismic loading conditions. Some common deficiencies of the then existing testing methods were found to be (Golder Associates 2002):

- single impact drop weight testing does not replicate cyclic seismic loading;
- the stiffness of the *in situ* loading system is generally not well replicated;
- bolts are usually tested only in tension and not in shear or combined shear and tension, although there are some exceptions. Underground observations show that a high

percentage of reinforcing elem ents can fail in shear under rockburst conditions (e.g. Haile 1999);

- the end fixity conditions and the constraints and confinement applied to a bolt in practice may not be replicated adequately;
- only the rock or cable bolt component is tested, not the rock mass-bolt system; and
- the carrier and rider waves reflected up and down the bolt in some drop weight systems (e.g. Yi & Kaiser 1994) are unlikely to have the same characteristics as the waves produced *in situ*.

In view of the increasingly severe service requirements of support and reinforcement systems and the importance of dynamically capable systems, it is hardly surprising that several papers to this symposium report the use of a range of static and dynamic laboratory and field tests on support and reinforcing elements (e.g. Aoki et al. 2004, Falmagne & Simser 2004, Gaudreau et al. 2004, Heal et al. 2004, Li et al. 2004, Player 2004, Player et al. 2004, Satola & Aromaa 2004, Thompson et al. 2004, Van Sint Jan & Cavieres 2004, Windsor et al. 2004). Gaudreau et al. (2004) provide a good review of testing methods and describe the quasi-static underground pull test system and the drop weight impact testing system used by Noranda.

The most advanced dynamic testing system known to the author is that developed recently at the Western Australian School of Mines (WASM), Kalgoorlie. The background, development, construction and initial application of this system are described by Player et al. (2004). Thompson et al. (2004) provide an analysis of the system that is implemented in a computer-based simulation. An important feature of the WASM dynamic test system that seeks to overcome at least one of the deficiencies of previous systems is that three components of the system representing the reinforcing element and the associated surface hardware, the rock ejected in a rockburst, and the surrounding rock mass, are dropped together onto an impact surface to generate dynamic loading of the system. Interestingly, the design uses what is described as the WASM momentum transfer concept (Player et al. 2004).

Rockbursts have been simulated by specially designed underground blasts to assess the dynamic performance of support and reinforcement elements and systems (e.g. Archibald & Katsabanis 2004, Haile & Le Bron 2001), and for other purposes. This approach is being used currently in a study of the performance of ground support systems subject to strong ground motion being carried out at a number of Western Australian mines that experience mining-induced seismicity and rockbursting (Heal et al. 2004). This program of testing is supported by an extensive array of monitoring equipment. Despite the advantages of this approach in carrying out well-designed and controlled *in situ* experiments, there remains the essential difficulty that the mechanics of blasting and the waveforms produced are not necessarily good representations of those associated with mining-induced seismicity.

4 ANALYSIS AND MODELLING

4.1 Classes of problem

In the analysis and numerical modelling of ground support and reinforcement for underground excavations in rock, several distinct classes of rock mass response may have to be allowed for:

- the sliding or falling of single, sometimes large, blocks of rock isolated by major discontinuities;
- the detachment of small blocks and the unravelling of the rock mass;
- beam action in laminated rocks;
- general shear (plastic) deformation of a zone of rock around the excavation;
- brittle fracture of the rock around (part of) the periphery of the excavation; and
- dynamic response to mining-induced seismicity.

Most of these classes of problem are represented in the papers presented to this symposium, although few of the papers report advances in analytical or numerical modelling capability. Only selected aspects of the broad topic of analytical and numerical methods will be considered here.

4.2 Analytical methods

Analytical solutions to simplified or idealised sliding block or wedge, roof beam and plastic zone problems are well-established in the rock mechanics literature (e.g. Brady & Brown 2004). However, somewhat reassuringly, improvements and extensions to established methods continue to be made (e.g. Carranza-Torres & Fairhurst 1999, Carranza-Torres et al. 2002, Chen 2004). In these solutions, the effects of support and reinforcement are usually allowed for only in a simplified way, as forces or pressures applied to the excavation boundary. Assumptions also have to be made about load distributions within the problem domain and the treatment of discontinuity normal and shear stiffnesses (Brady & Brown 2004). Although there have been some heroic attempts to model rock bolt behaviour analytically (e.g. Indraratna & Kaiser 1990), more complete solutions to dynamic support and reinforcement problems are even more simplified. They usually involve energy dissipation calculations based on an assumed velocity of ejection of fractured rock from the surface of the excavation.

Among the papers to this symposium, block stability analyses for the El Teniene Mine, Chile, are reported by Bonani et al. (2004). As is common practice, software packages were used to obtain solutions. Belem et al. (2004) provide methods of design analysis for a number of aspects of the stability of paste fill walls and working surfaces. Gaudreau et al. (2004) present a method of calculating the displacement of a tendon subjected to impact loading based on a critically damped harmonic motion model incorporating a "friction factor" and a yield point offset. Rojas et al. (2004) provide details of energy absorption calculations used for rockburst conditions at El Teniente Mine, Chile.

4.3 Numerical modelling

Numerical analysis of both continuum and discontinuum problems in rock engineering is now well established. Jing (2003) recently provided a valuable review of the techniques available and the outstanding issues associated with numerical modelling in rock mechanics and rock engineering. Interestingly, Jing's review made little mention of the incorporation of support and reinforcement into the wide range of numerical methods now available. The most useful methods available for this purpose known to the author are the methods of modelling reinforcement due to Brady & Lorig (1988) incorporated into the finite difference codes FLAC and FLAC3D. Models are available for both local reinforcement (or individual reinforcing elements) and for spatially comprehensive reinforcement. However, even these models involve a number of assumptions and idealizations and do not model accurately all aspects of the observed responses of reinforcement elements and systems.

Numerical modelling is used in a number of papers presented to this symposium. Aoki et al. (2004) adapt Brady & Lorig's (1988) model to Swellex friction anchored rock bolts. Seedsman (2004) uses the Phase² plastic finite element model to elucidate a number of aspects of the failure modes of coal mine roofs under varying imposed stresses. Silverton et al. (2004) describe how sometimes quite sophisticated non-linear numerical modelling is being used as part of a risk-based design approach in civil engineering tunnelling. Wiles et al. (2004) present a procedure for the design of reinforcement for highly stressed rock based on numerical stress analysis using the MAP3D elastic boundary element code and illustrate the method's application to underground mining in hard rock. Thibodeau (2004) reports the application of MAP3D and the wedge analysis program UNWEDGE in studies of the support and reinforcement of intersections at the Creighton Mine, Canada.

4.4 Ground-support interaction analyses

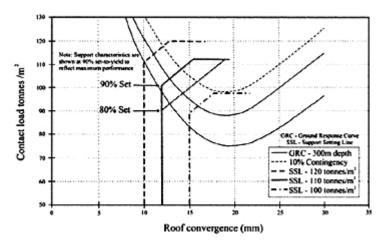
Although ground-support interaction analyses have existed conceptually for several decades, they appear to have found increasing use in a range of applications in recent years. As well as the general or indicative uses such as that shown in Figure 1, ground-support interaction diagrams have been calculated analytically and numerically for a range of design problems. Carranza-Torres & Fairhurst (1999) showed how FLAC3D may be used with a Hoek-Brown yield criterion to calculate ground reaction curves and the extent of plastic zones around advancing tunnel faces. Leach et al. (2000) provided an instructive example of the use of FLAC3D in the calculation of ground reaction curves and their application in the design of extraction level excavations in the Premier Mine, South Africa. The curves were used to evaluate the levels of support pressure required to limit drift closures to acceptable levels for a number of scenarios.

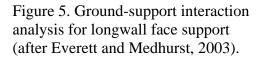
More recently, Everett & Medhurst (2003) reported the successful application of the ground response curve method to a number of Australian longwall coal mines. Figure 5 shows calculated ground characteristic lines or ground response curves (GRC) for typical Australian longwall conditions for a depth of 300 m and allowing for a 10% additional loading contingency for a given convergence. Support characteristics are shown for installed chock loading capacities of 100, 110 and 120 t m⁻². These characteristics are shown with a 90% ratio of setting load to yield load to reflect optimal performance. In

one case, a 80% setting to yield load ratio is also shown. As shown by Figure 5, underrated supports (in this case the 100 t m^{-2} support) may allow excessive convergence before being set, and may not be able to accommodate the full load generated once deterioration of the roof develops.

4.5 Brittle fracture

Although not related specifically to the modelling or design of support and reinforcement systems,





a significant advance has been made in recent years in the modelling of brittle rock fracture around underground excavations. There is believed to be considerable potential for the further application of the method developed by Martin (1997) and Martin et al. (1999).

In laboratory and field and field studies of the behaviour of Lac du Bonnet granite, Martin (1997) found that the start of the fracture or failure process began with the initiation of damage caused by small cracks growing in the direction of the maximum applied load. For unconfined Lac du Bonnet granite, this occurred at an applied stress of 0.3 to 0.4 σ_c where σ_c is the uniaxial compressive strength of the intact rock material. As the load increased, these stable cracks continued to accumulate. Eventually, when the sample contained a sufficient density of these stable cracks, they started to interact and an unstable cracking process involving sliding was initiated. The stress level at which this unstable cracking process is initiated is referred to as the long term strength of the rock, σ_{cd} .

As illustrated in Figure 6, Martin (1997) first determined the laboratory peak, long term and crack initiation strengths for the Lac du Bonnet granite. He was able to fit

Hoek-Brown failure envelopes to these curves, although the laboratory crack initiation curve was found to be a straight line on σ_1 versus σ_3 axes. Subsequently, in a field experiment carried out at the Underground Research Laboratory site in Manitoba, Canada, the initiation of cracks around a tunnel excavated in the Lac du Bonnet granite was recorded using microseismic emissions. As shown in Figure 6, these

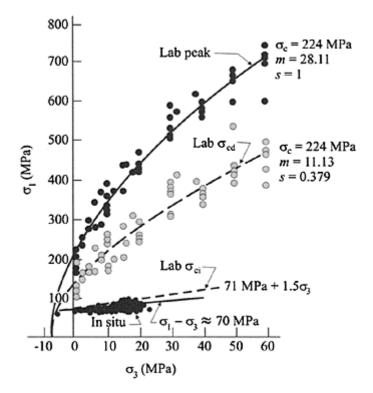


Figure 6. Hoek-Brown failure envelope for Lac du Bonnet granite based on laboratory peak strength (Lab Peak), long-term strength (Lab σ_{cd}) and *in situ* crack initiation stress (σ_{ci}) determined by microseismic monitoring (after Martin 1997).

data corresponded well with the laboratory crack initiation data. It was found that crack initiation at approximately constant deviatoric stress, ($\sigma_1 - \sigma_3$), could be well represented by the Hoek-Brown criterion with m_b=0 and s=0.11 (Martin et al. 1999), in which case, $\sigma_1 - \sigma_3 = 0.33\sigma_c$.

This criterion was used in conjunction with elastic stress analyses to give good predictions of the geometry of the spalled zone around the tunnel. It has since been used to predict brittle spalling or slabbing (as opposed to general shear failure) conditions in a number of underground excavations (e.g. Cai et al. 2004, Rojat et al. 2003). In effect, this criterion is similar to the Rock Mass Damage Criterion discussed by Wiles et al. (2004).

Brown (2004) has suggested that this criterion could also be used in analyses of the likelihood of brittle rock fracture around highly stressed extraction level excavations in block and panel caving mines. In addition to the uniaxial compressive strength of the rock material, the state of stress on the boundary and in the rock around particular excavations would have to be estimated. This can be done through a three dimensional elastic stress using the finite difference code, FLAC3D. Depending on the geometry of the problem to be investigated, two dimensional plane strain analyses may be used in some cases.

It has been found that the loading path taken to the current state of stress can influence the strength able to be developed by a rock mass. This is particularly the case when plastic deformation is involved. However, when deformation is essentially elastic until "failure" and brittle fracture as opposed to general plastic deformation occurs, it has been found that the strength envelopes of "hard" rocks are essentially stress path independent (Brady and Brown 2004). It is considered likely, therefore, that as in the cases described by Martin et al. (1999) and Cai et al. (2004), elastic stress analyses and Martin's representation of the Hoek-Brown criterion for crack initiation will suffice for making a first-order estimate of the occurrence and extent of brittle fracture around extraction level excavations in strong, massive rock.

5 OVERALL SUPPORT AND REINFORCEMENT PERFORMANCE

The evidence contained in papers presented to this symposium and elsewhere, suggests that support and reinforcement systems are now being provided successfully under a range of extreme service conditions including highly stressed or squeezing ground (e.g. Button et al. 2003, Tyler & Werner 2004), rockburst conditions in a range of underground mines (e.g. Dunn 2004, Falmagne & Simser 2004, Rojas et al. 2004) and brittle fracture around deep tunnels and other civil engineering excavations (e.g. Cai et al. 2004, Rojat et al. 2003).

Despite the advances that have been made, rock-falls remain one of the major causes of injuries and fatalities in underground mines (Potvin et al. 2004). Lang & Stubley (2004) summarize an extensive data base on rockfalls in Western Australian underground mines for the period 1980 to 2003, and a range of legislative and other measures taken to reduce the rockfall hazard in Western Australia.

Potvin et al. (2004) report the results of an important study of the effectiveness of reinforcement and surface support in controlling rockfalls in Australian underground metalliferous mines. They found that there are short-comings in the timing of the installation of reinforcement and in the use of surface support. The risk of injury from rockfalls in greatest near the working face where mining activities are intense and workers are exposed to unsupported faces and walls. Rockfalls do occur away from working faces but only a small proportion of them cause injury. Despite this finding, it is also the case that most recent rockfall fatalities have involved large falls from supported ground more than 50 m away from an active face (Potvin et al. 2004). It is clear that,

despite the advances that have been made, more needs to be done, particularly to reduce the exposure of personnel to working faces.

Szwedzicki (2004) argues that the use of quality assurance procedures in ground control management programs contributes to improvements in the safety and productivity of underground metalliferous mines. Dunn (2004) reports the development of ground support evaluation and quality assurance procedures for AngloGold's South African region. This was in response to a new mining regulation introduced in South Africa from January 2003 which provided that "at every underground mine where a risk of rock-bursts, rock falls or roof falls exists, the employer must ensure that a quality assurance system is in place which ensures that the support units used on the mine provide the required performance characteristics for the loading conditions expected".

6 CONCLUSIONS

It is clear from the brief review reported here that several significant advances have been made in support and reinforcement practice for underground excavations in the five years since the last symposium in this series. The advances made include:

- the development of new and improved rock and cable bolt elements;
- greater use of fully encapsulated rock bolts;
- improved one pass bolt installation systems;
- the introduction of hydro-scaling and in-cycle scaling and support installation;
- an improved understanding of corrosion and corrosion protection, particularly of reinforcing elements;
- the development and use of new types of mesh and spray-on liners;
- the increased use being made of shotcrete, particularly wet mix fibre-reinforced shotcrete;
- the increased use being made of paste fill in cut-and-fill, bench-and-fill and open stoping methods of mining;
- the development and successful application of dynamically capable support and reinforcement systems;
- the development of new and improved dynamic testing systems;
- incremental advances in some aspects of analytical and numerical modelling capability;
- some impressive achievements in ground control under demanding service conditions in both underground mining and civil construction;
- improved understanding of the causes of rockfalls in underground metalliferous mines developed as a result of a program of intensive data collection and analysis; and
- the greater use being made of quality assurance procedures in ground control management programs.

Almost all of these advances are represented in the papers presented to this symposium. Despite the continuous improvement being made, there remains a need and the scope for further improvement and new developments. Some of the issues that appear to the author to be particularly pressing are:

• further development of in-cycle scaling and support and reinforcement installation systems;

- as part of this, the development of mechanised installation methods (e.g. for the new types of mesh and TSLs being trialled) with a view to minimising the exposure of personnel to unsupported ground near an advancing face;
- the exercise of even greater control over the quality of support and reinforcement materials and installation processes; and
- further research into the characteristics of mining-induced seismicity and the associated development of improved, more fundamentally based, analysis and design methods for dynamic conditions.

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A review of long, high capacity reinforcing systems used in rock engineering

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ABSTRACT: Cable bolts have provided the mining industry with increased production, safety and flexibility in production excavations. However, with the development of wider span haulage and other, larger, mine openings, cable bolts are now also used to stabilize longer life, infrastructure excavations. In these circumstances, the requirements of stability and longevity are more important than for non-entry excavations. This discussion will explore some of the issues that affect the quality, performance and longevity of cable bolts.

1 INTRODUCTION

Cable bolts are long, high capacity reinforcing systems. A review of the historical development of cable bolts and the associated technology has been attempted by Windsor (1992). That review found that cable bolt technology is related to ground anchor technology and that both disciplines use devices and techniques originally developed for pre-stressed concrete. The common theme is the use of 7-wire pre-stressing strand, its internal fixture using cementitious grouts and its external fixture using pre-stressing grips. In all three applications, these products (i.e. strand, grout and grips) are used to produce a tensile contribution in materials that are characteristically weak in tension (i.e. concrete, rock and soil). However, there are marked differences between the disciplines in the aims and operation of these products.

In concrete flexural members, they are pre-stressed to pre-compress the member such that in service, during flexure, the concrete acts predominantly in compression. In ground anchors they are also pre-stressed and are used to both, anchor structures (e.g. dams, diaphragms) and to reinforce and support soil and rock masses. In cable bolting, like ground anchoring, they are also used to reinforce and support soil and rock but with relatively little or no pre-stress. The main aim has been to reinforce (as opposed to pre-stress) the rock, as is the case in reinforced concrete engineering. The fact that a pre-stressing element is used as reinforcement is an important distinction in that the stiffness and elongation requirements in these two modes are different. This distinction is seen in the evolution of cable bolts (Figure 1) where the 'degree' and 'extent' of coupling the strand through the grout to rock has been modified. These modifications provide a range of 'reinforcement' systems with the stiffness and elongation characteristics required for different levels of rock mass stiffness and deformation.

An excellent review and summary on ground anchor technology was presented at the same conference by Littlejohn (1992). A comparison of the two reviews shows marked differences associated with the quality of material components, the quality of

workmanship in the assembly of components into systems and the formal requirement of proof testing for all installations. This reflected the fact that in mining, cable bolts were predominantly being used for temporary, low entry excavations where ground anchors are used for civil infrastructure. Nevertheless, at workshops held in conjunction with that same conference, the consensus was that cable bolting practice could and should be improved by collating international experience and preparing a handbook for use by mining engineers. Consequently, a project was initiated and conducted by the Geomechanics Research Centre at Laurentian University, Canada and the CSIRO Rock Reinforcement Group, Australia in order to consolidate and compile joint experiences gained in cable bolt research and development. This project was funded by 23 mining companies through the Mining Research Directorate of Canada and the Australian Mineral Industries Research Association and produced the handbook 'Cablebolting in Underground Mines' by Hutchinson & Diederichs (1996).

Significantly, the handbook is descriptive, not prescriptive, presenting the key practical aspects of cable bolting in such a manner that enables companies to base their site-specific designs and procedures on good practice without restriction. There is no doubt that cable bolting practice has improved in all countries over the last decade as a result of that work and the many international workers who have contributed their experiences. The book is recommended to all mining companies who conduct cable bolting.

Some ten years later, we now find that cable bolts not only continue to provide excellent service in production excavations but are also used to secure many mine infrastructure excavations, some of which are large. Furthermore, the details that have concerned civil engineers for many years (e.g. quality control, workmanship, durability, corrosion etc.) are now, also of concern to some mining engineers. Fortunately and, as a consequence of following behind ground anchor technology, cable bolt practice can benefit from the hard won lessons and advances made by our civil engineering colleagues. This valuable information is available in the form of:

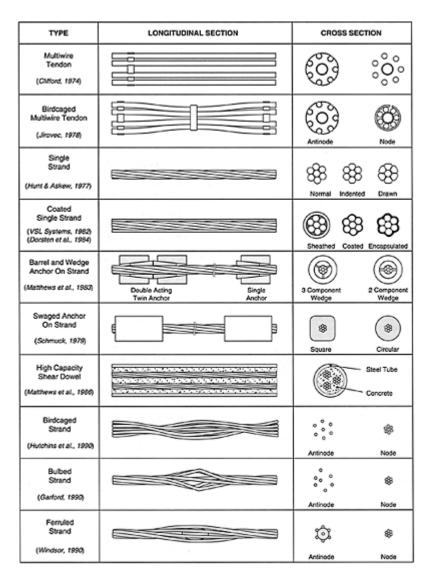


Figure 1. The evolution of cable bolt devices to provide various levels of stiffness and elongation.

- Conference papers. (e.g. The International Symposium on Anchors in Theory and Practice held in Salzburg in 1995, The ICE Conference on Ground Anchors and Anchored Structures held in London in 1997.)
- State of the art reviews. (e.g. Littlejohn & Bruce 1977, Fuller 1983, Barley 1988, Windsor 1992, Littlejohn 1992, Bruce 1993, Weerasinghe & Adams 1997, and Barley & Windsor 2000)

- Books. (e.g. Hanna 1982, Hobst & Zajic 1983, Habib 1989 and Xanthakos1991)
- Codes of practice. (e.g. CIRIA 1980, ONORM 1985, DIN 4125 1988, BS 8081 1989, SAICE 1989, PTI 1996, FIP 1996, and EN1537 1996)
- Material compliance standards. (e.g. Australian standards AS 1310, AS 1311, AS 1312 and AS 1314 from Standards Australia 2004, American Standards for Testing and Materials ASTM A421, ASTM A722 and ASTM A416 from ASTM 2002.)

This discussion will refer to this information and use the reinforcement system concept to explore some important subtleties of cable bolt system design and how these affect the installation and performance of cable bolts. The fact that pre-stressed concrete and ground anchor technology is regulated by the civil engineering industry and that cable bolt technology is not, is central to the future of cable bolt practice. The intention here is not to propose regulation but to highlight issues that have eventually required regulation in other industries.

2 THE MECHANICAL PROPERTIES OF REINFORCEMENT AND SUPPORT

The most important set of mechanical properties for reinforcement and support are those that define its service behaviour in situ. Clearly, this depends on the properties of the parts that make up the system and the interaction of the parts. A simple system representation that suits rock reinforcement, and thus cable bolts, comprises four components:

- The Rock.
- The Element (Strands).
- The Internal Fixture (Grout).
- The External Fixture (Plate and Grips).

There are many important properties associated with each component that affect its behaviour and how it interacts with the other components. In civil engineering industries the properties of the components and their assembly into load bearing systems are regulated by material compliance standards, design codes of practice and finally in situ przfsoof testing. In mining engineering application of cable bolts there are no such formal requirements. Moreover, the competitive nature of the mining industry means that there is a continual need to reduce costs. Unfortunately, this imposes financial pressures on the manufacturers/suppliers of products and imposes installation difficulties on mine personnel.

A classic example is the quality and operational characteristics of some rock bolt expansion shell anchor designs used in the mining industry. Originally, the sliding parts (the wedge and shell) were provided with machined and lubricated surfaces. Once, this was considered a critical design feature of expansion shell anchors. However, these anchors can be also be produced by casting, rather than machining the parts. This results in much higher levels of friction between the sliding parts which affects the installation and finally, the operation of the rock bolt system. It has been found that these anchors, dictate the load-displacement response of the system with ultimate load capacity falling well below that of the element. The apparent costs savings are insignificant in comparison to the overall cost of drilling the hole and installing the device. Further, two devices have to be installed to achieve the capacity of one with a slightly more expensive anchor. This is an example of how sub-optimal system behaviour can be imposed on the mining industry by the mining industry in pursuit of an apparent cost saving.

2.1 Mechanical properties in regulated industries

The engineering disciplines that conform to design codes of practice and material compliance standards often consider five levels for a particular mechanical property (e.g. load capacity of a reinforcing element):

- 1. Intrinsic Value (V1). This is the true capacity of the component.
- 2. Laboratory Test Result Value (V2). This value may be affected by the 'system' configuration used to test the component.
- 3. Standard Minimum Required Value (V3). This is the minimum acceptable intrinsic value stipulated by a 'material compliance standard'.
- 4. In Situ Test Result Value (V4). This value may be affected by the 'system' configuration used in situ.
- 5. Maximum Service Value (V5). This is the maximum value allowed to be reached in service, stipulated by some 'design compliance standard'.

Usually, but not always, $V1 \ge V3 \ge V3 \ge V4 \ge V5$. As an example, consider testing for the axial capacities of 7-wire steel strand of a given nominal diameter. In pre-stressed concrete material compliance standards, V3 is often set well below V1 such that when manufacturers obtain V2 for their product, it has every chance of satisfying V3. However, V2 is dependent on the testing arrangement (i.e. a system). In a laboratory axial load test, the strand (element) is held at each end in a clamp or vice (fixture). The vice length and surface geometry are configured to ensure that premature rupture of peripheral wire(s) is not initiated by stress concentration notches imposed by say, the vice teeth. The desired failure is a 'cup and cone' shaped rupture surface in one or more wires situated at a minimum number of diameters away from the vice at V2=V1. In some test arrangements, premature failure may occur at the vice as a complex tensile, torsional, shear rupture at a point of stress concentration initiated by the vice. Regardless, in most cases of premature failure of strand, V2 (load) is still >V3 (load) and thus the strand still satisfies the minimum required load capacity, even though it has not been tested properly.

Although it is relatively simple to satisfy a material compliance standard in terms of load capacity the same is not true for elongation capacity. For example, samples of 7-wire strand have been tested to achieve over 13% elongation of the element. With other test arrangements, the fixture (e.g. barrel and wedge anchor or 'grip') initiates premature failure. The 'test' may well satisfy the load capacity requirement, but the strand may not reach the relatively small elongation requirement (e.g. 3.5% in AS 1311, ASA 1987) even though its intrinsic value is substantially greater. In this example V2 and V4 are dependent not only on the intrinsic properties of a single component (the strand) but more specifically, on the interaction of two components—the strand and the grip.

The expansion shell anchor and strand/grip examples were chosen to illustrate the negative case (i.e. full capacity of the element is not utilised due to the behaviour of one or the interaction of two components in the system). This important observation can be reversed to produce the positive case. That is, the system can be designed to extract

maximum load and displacement capacity from the element without inducing its 'failure' (see Windsor 1996).

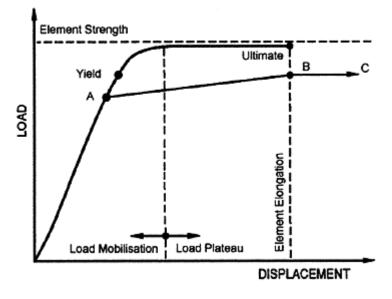


Figure 2. Optimal system behaviour for a given element.

Consider Figure 2, which shows the intrinsic load displacement behaviour of some element as it mobilises load through yield to its ultimate. Basically, we seek to arrange the system such that its behaviour 'tracks' the intrinsic, 'elastic', response of the element from 'O' to a point 'A' and then follows some 'plastic' response path through 'B' to 'C' (just below element yield). For material with a limited elongation capacity, optimal system performance is obtained by arranging for unlimited displacement in the system at 'just below' yield of the element.

2.2 Compliance issues for reinforcement in an unregulated industry

In regulated industries, in situ proof testing of the system has the distinct advantage of simultaneously testing all the components, their interactions and the quality of the installation. In comparison, an unregulated, cost competitive industry must ensure that path ABC and onwards, is not limited by non-compliance of material properties and suboptimal design of systems. This allows mine personnel to concentrate on installation practice. This is especially true if systems and procedures developed some 20 years ago for temporary purposes are to be used to secure semi-permanent mine infrastructure.

The following sections will explore the requirements of the principal components of the system and their interactions together with two issues of installation, that affect performance and longevity: pretensioning the system and corrosion of the system.

3 THE ELEMENT

The intrinsic material properties of steel reinforcing elements depend on strict adherence to a carefully controlled thermo-chemical process to produce the steel melt and then a carefully controlled thermo-mechanical process to produce the element to specific dimensions and achieve minimum mechanical properties. Material compliance standards are imposed on reinforcing elements on a national basis to ensure that they meet minimum mechanical specifications in service. For example, in the USA, stress relieved wires must conform to ASTM A421, stress relieved strand to ASTM A416 and high tensile bars to ASTM A722.

In late 2003, the USA Department of Commerce announced antidumping duties on strand supplied at less than fair value to the USA market in contravention of International Trade Law. The duties imposed indicate the cost differential on fair price and were listed as: Brazil (119%), India (103%), Mexico (77%), Korea (54%) and, Thailand (12%). A discussion on the quality of one such product is given by Concrete Products, 2003.

Free international trade is advantageous if cheaper imported products can be obtained that satisfy national standards. Currently, Australia is only able to produce about 50% of its national usage of 7-wire steel strand. The sources of the remainder are difficult to define. Fortunately, there are two developments underway in Australia that aim to ensure proper manufacture and compliance of reinforcement products for use in prestressed concrete and ground anchors.

Firstly, three Australian standards have recently been revised by Standards Australia (SA), namely:

- Steel prestressing materials (SA 2004). This is a draft revision of AS 1310–1987; AS 1311–1987 and AS 1313–1989.
- Prestressing anchorages (SA 2004). This is a revision of AS 1314-1972.
- Galvanized steel wire strand (SA 2004). This is a draft revision of AS 2841-1986.

Secondly, the Australian Certification Authority for Reinforcing Steels (ACRS) has been set up as the national facility to test and certify compliance for all reinforcing steel products used in Australia. At this stage, no strand products have received ACRS certification (ACRS May, 2004, pers. comm.).

It is worth noting that, in the past, reinforcing elements have been imported into Australia for use in the mining industry that have fractured when dropped onto bitumen from waist height, others have been found with inclusions in the steel microstructure. Some strand has been found to retain a sinusoidal shape 'memory' of coil size, which affects both installation ability and its profile in the borehole. Some strands have also been found to be dimensionally inconsistent and others with strange rupture characteristics. These products indicate that sub-standard steel manufacture and or element forming processes and or heat treatments have been used.

It is recommended here, that the steel reinforcing products used in the Australian mining industry be certified to comply with the new 2004 Australian Standards. Clearly, non-standard or unidentified materials should not be used for reinforcing elements. Any cost advantage that might be declared is accompanied with the attendant risk of material failure, its subsequent cost and the possible legal consequences. The following section

will explore the general requirements of the strand used in cable bolts and why this recommendation is given.

3.1 Steel manufacture

The mechanical performance of the steel used as a reinforcing element is controlled by the various chemical, thermal and mechanical treatments used to produce the element. Different grades of steel may be produced by varying two aspects in the manufacturing process, the composition and amount of chemical constituents in the steel melt and the thermo-mechanical procedure to work the melt into the final product.

The constituents of steel and their maximum proportions are restricted by National Standards. Commonly, the constituents and proportions are Carbon (0.25%), Phosphorous (0.04%), Manganese (1.50%), Silicon (0.50%), Sulphur (0.04%) together with trace amounts of Nickel, Chromium, Copper and other grain refining components. High yield stress or high strength steels are created by 'carefully' increasing the Silicon content and the trace amounts of Chromium, Copper, Nickel and Vanadium.

3.2 Element manufacture

Once the steel melt has solidified, it is 'worked' to form a shape but this work can modify its mechanical properties. For example, if the steel is deformed beyond its elastic limit, unloaded and then reloaded, its elastic limit will be increased. This is termed 'strain hardening' and may be used to improve the elastic limit but it is also accompanied by a reduction in elongation capacity (or ductility). At high temperature some of the effects of strain hardening can be recovered or relieved and this process is known as 'hot working'. At temperatures that do not allow strain hardening to be relieved the process is termed 'cold working'.

Cold working is used to produce the steel elements used in reinforcing practice where the steel stock is sequentially reduced in cross sectional area by drawing it through a sequence of reducing dies. The cold worked steel may then be subject to further heat treatment to bring about changes in the microstructure and desirable mechanical properties. Typically, reinforcing elements may be produced with yield stresses that range from 200 to 2000 MPa depending on this manufacturing process.

As an example, consider Figure 3 which compares the relationship between element diameter and the minimum required yield force standards for reinforcing bars, prestressing wires and pre-stressing strand, all of which have been used as rock reinforcement elements.

3.3 Axial force capacity of steel strand

The steel element commonly used for cable bolts is 7-wire steel pre-stressing strand. This strand comprises a slightly larger diameter, central, 'king' wire and 6 outer or 'peripheral' wires in a left or right lay, helical packing configuration. In the 15.2 mm diameter variant the individual wires are individually cold drawn down to diameter of approximately 5 mm before spinning to form the strand. Depending on manufacture, the 15.2 mm diameter variant may be produced to provide a yield load capacity of between

190 and 260 KN and an ultimate load capacity of between 220 and 300 KN. The grades of strand available include Regular, Super, Extra High Strength and Compact where each grade is required to satisfy minimum mechanical properties.

A comparison of minimum yield force and minimum breaking force for different grades of steel strand available in Australia are given in Table 1.

3.4 Elastic moduli of steel strand

The modulus of elasticity of most steel (E_s) is reasonably constant at around 200 GPa with rolled, stretched and tempered steel bar at 205 GPa and steel prestressing wire and strand at about 195 GPa. In all cases, there is a reduction of modulus with length as shown by Janiche (1968) who found a 7.7% reduction in E_s for elements tested over 100 m and by Leeming (1974) who found a reduction of 9.2% in E_s for lengths longer than 36 m. British Standard BS 8081 (1989) states that small lengths of strand tested in the laboratory indicated E_s of 180 to 220 GPa whereas longer lengths tested in situ indicated E_s of 171 to 179 GPa.

For most steels, Poisson's ratio ranges from 0.25 to 0.29. However, the composite nature of strand means that its lateral contraction under axial load is greater than solid wires and bars. This is because the peripheral wires, in an attempt to straighten out under load, rotate in the lay direction, effectively packing the wires closer together and reducing the overall strand

Strand grade	Min. yield force (kN)	Min. breaking force (kN)
Regular	193 (100%)	227 (100%)
Super	213 (110%)	250 (110%)
Extra High Strength	222 (115%)	261 (115%)
Compact	255 (132%)	300 (132%)

Table 1. Different grades of 15.2 mm diameter steel strand.

Note that the minimum requirements vary by up to 32% depending on the specified grade of strand.

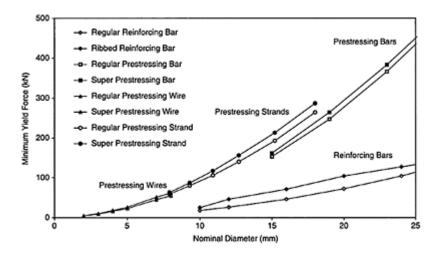


Figure 3. Reinforcement minimum yield force requirements.

diameter. Compact strand has a lower lateral contraction than plain strand.

In situ, in grout and under axial loads, uncompacted strand laterally contracts away from the grout interface, effectively reducing the components of adhesion, mechanical interlock and friction. This may cause the propagation of a debonding front along the element grout interface, which invalidates a linear extrapolation of load transfer characteristics from shorter to longer lengths.

This phenomenon of a yielding interface is useful in that it may be manipulated reduce axial stiffness. However, if the strand is un-tensioned prior to grouting, the wire must be uniformly packed. The effects of excessive contraction under initial load can greatly affect the constitutive relationship for bond failure (e.g. Hyett et al. 1995).

3.5 Axial elongation capacity of steel strand

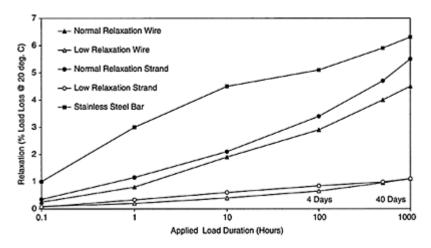
The axial strain capacity of reinforcing elements is also addressed by standards. For example, in Australia, reinforcing bar requires an ultimate elongation ε_{ult} of >22%, prestressing bar requires ε_{ult} of >5% and pre-stressing wire and strand requires ε_{ult} of >3.5%. It is important to note that a reinforcing element for use in reinforced concrete is required to satisfy a minimum elongation that is 7 times more than the minimum ultimate elongation required of a pre-stressing element.

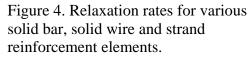
Prestressed concrete and ground anchors do not require high elongation characteristics of elements because in these disciplines the structure is designed to operate at low levels of deformation with strands arranged to operate 'well below' yield (A in Figure 2). In comparison, cable bolts are expected to reinforce and support a rock mass, which may crack and deform significantly, requiring considerable elongation (ABC in Figure 2). If this elongation cannot be supplied prior to element yield, the system must be softened by modifying the way the element is coupled to the rock. For example, cable bolt systems may be decoupled over a given length to provide the required elongation or may be arranged to slip at loads below yield of the element (see Figures 1 and 2). These cases are examples of elastic and plastic 'system' behaviour respectively. In both cases, load-displacement must ultimately be controlled by other components in the system for both yield and uncontrolled slip of the element are undesirable.

3.6 Relaxation and creep of steel strand

Relaxation is a reduction in load under conditions of fixed displacement. Creep is an increase in displacement under conditions of fixed load.

Loss of load will occur in stressed steel due to relaxation with time and is temperature dependent, relaxing more at higher temperatures. Relaxation is usually expressed as the X% loss of force in the element at 20°C in a 1000-hour period when the element is stressed to 70% of its characteristic strength. There are two relaxation grades of steel bar, wire and strand termed normal (N) and low (L) relaxation. A comparison of the load relaxation of various rock reinforcement elements with the duration of loading is given in Figure 4.





In design practice, the maximum relaxation is determined using multipliers to account for the load loss depending on the permanence of the system. BS8081 1989 suggests factors of 1.5 and 2.0 for permanent N and L systems respectively and 1.25 and 1.75 for temporary N and L systems respectively. Further, multiplier coefficients are also suggested for service at elevated temperatures, these are $1.0 @ 20^{\circ}$ C up to $2 @ 40^{\circ}$ C. It is important to note that these relaxation rates relate solely to the inherent properties of the element (i.e. its composite nature and its microstructure).

Creep tests are much more difficult to perform than relaxation tests due to difficulties associated with applying a constant high load. Creep experiments conducted at the NBHC/Zinc Corp. mine in Broken Hill by CSIRO in the early 1980's involved creep testing of 15.2 mm strand installed in cementitous grout within split, 2 m long, thick walled, steel tubes. The upper tube was locked in position, the lower being loaded by a dead weight. It was found that at about 200 kN, the strand was statically stable but crept with increasing time and after a few months developed a rotational bond failure leading to pullout. Here, creep of the system was assessed which may be affected by relaxation of the element.

In summary, the load displacement capacities, stiffness (Es), relaxation and creep and the packing of wires affect consistent response. Relaxation and creep combine with radial contraction to become critical in initially un-tensioned systems. In regulated industries, some of these problems are minimised by cyclic loading during proof testing. In untensioned systems these phenomena occur during the process of reaching the in situ load. Latter we will discover that consistent element diameter is also important for consistent 'system' behaviour.

4 THE INTERNAL FIXTURE

A reinforcement element may be fixed to the rock within the borehole by mechanical or chemical internal fixtures. Mechanical fixtures include expansion shells and grips, both fix the element to the rock at a point. Chemical fixtures include polymeric and cementitious grouts both fix the element to the rock over a continuous length. Polymeric grouts are more suited to shorter length devices where placement problems are not so severe. In long installations, cementitous grouts are the standard internal fixture in both civil and mining rock reinforcement.

Discussions and the results of research work conducted on cementitious grouts for use in ground anchors has been given by Littlejohn (1982) and Barley (1997) and for use in cable bolts by Hyett, et al. (1992) and Thompson & Windsor (1999). This discussion will explore the effects of the various constituents of grout on its performance and durability.

Cementitious grouts basically comprise cement (Portland Cement) and water mixed in various water cement (w/c) ratios and can also include admixtures to improve certain chemical or physical properties. The use of cementitous grout for application with reinforcement involves four issues:

- Selection of materials (cement, water and if neccessary, admixtures).
- Transport and storage of the materials.
- Metering and mixing of the materials.
- Placement of mixed materials in the borehole.

Any variations in the quality or quantity of materials and departures from accepted mixing and placement practices will affect the mechanical properties of the grout in its fluid and hardened states and will therefore affect the final reinforcement system.

The requirements of the grout in its fluid and hardened state often conflict; the former concerns proper placement of the fluid, the latter concerns achieving appropriate mechanical properties.

4.1 Requirements of cementitious grouts

The general prerequisites of grout in its fluid state enable it to be properly placed within the borehole without diminishing the required properties in its hardened state. The fluid state requirements include:

- Homogeneity.
- Low air entrainment.
- Low bleed characteristics.
- Appropriate set and hardening times.
- Appropriate pumping characteristics.

The final hardened state requires mechanical properties that allow the grout to interact with the rock and the reinforcement and chemical properties that allow it to maintain form and resistance to attack by deleterious agents within the rock mass. The hardened state requirements include:

- Homogeneity.
- High axial and shear strength.
- High axial and radial stiffness.
- High compaction at bond interfaces.
- High alkaline environment.
- High chemical resistivity.
- Low porosity and permeability.

Satisfying these requirements depends largely on using fresh cement, uncontaminated water, the correct w/c ratio and appropriate equipment for mixing and pumping the grout.

4.2 Cement types

Most cement is based on an inorganic binding agent made from a mixture of calcium carbonate, silica, alumina and iron oxide, which is fired in a kiln then finely ground. The resulting powder is known as hydraulic cement (i.e. capable of being mixed and pumped and of setting and hardening in the presence of water). Basically, there are two classes of cement, Portland cement and blended cement. Blended cements are mixtures of Portland cement and other materials that possess either inherent cementitious properties (e.g. Blast Furnace Slag) or pozzalanic properties that form cementitious compounds when mixed with water (e.g. Silica Fume and Fly Ash).

The main constituents of Portland cement comprise silica, alumina, ferric oxide, lime, magnesia and sulphuric anhydride; minor constituents include soda, potash and chloride. These are arranged in four main compounds: tricalcium silicate, dicalcium silicate, tricalcium aluminate and tetracalcium aluminoferrite. Tricalcium silicate is the most desirable compound because it hardens quickly and produces high early strength; dicalcium silicate develops strength slower but contributes to final cured strength.

Both Portland Cement and blended cements are made with specific proportions of constituents to exacting standards that demand minimum compressive strengths as shown in Table which is compiled from details given in AS 3972—Portland and Blended Cements (ASA 1991). Further detail can be obtained on blast furnace slag from Hinczak 1991, on fly ash from Ashby 1990 and on condensed silica fume from Papworth (1992).

Five cement types are specified in AS 3972, their coded type and strengths are given in Table 2. Here columns A and B give the minimum compressive strengths required of the cement mix after 7 and 28 days respectively.

GB cement is not recommended for use with cable bolts. It's minimum strength requirements are lower than that of GP cement. HE cement develops strength more rapidly due to its finer particle distribution and increased specific surface area for hydration (discussed later). It does not necessarily set quicker but does possess a higher rate of heat evolution. LH cement

Cement	Туре	A (MPa)	B (MPa)
General purpose	GP	25	40
Portland			
General purpose	GB	15	30
Blended			
High Early Strength	HE	30	30
Low Heat	LH	10	30
Sulphate Resisting	SR	20	30

Table 2. Cement types specifications in AS 3972 (ASA conditions.1991).

Note. AS 2350.4 requires a minimum and maximum set time of 45 mins and 10 hours respectively for each type.

is formulated to reduce heat evolution in order to limit thermal stresses. SR cement is formulated for grouts that need to operate in environments with sulphate ground waters. Other cement types exist and some have desirable properties but also, other short-comings. For example, High Alumina Cement possesses rapid strength gain, high early strength and resistance to sulphate attack but suffers substantial loss of strength in warm (>25°C) and humid conditions.

It is recommended that GP cement be used for all rock reinforcement installations except where conditions call for cement with special properties. However, it is critical to note that the relative quantities of the four major compounds and the particle distribution curve may vary between batches and between manufacturers depending on the source of the materials and the firing, grinding process. Consequently, testing should be conducted at regular intervals to establish the correct w/c ratio for each different source of GP cement used on site.

4.3 The cement particle size distribution

The fineness of the cement powder is controlled by the grinding process and affects both setting time and strength. The smaller the particles, the more chance that particles can be involved in hydration to produce gel and crystalline products. Consequently, fine cements produce more rapid hardening and stronger grouts than cements with coarser particles.

The particle distribution curve is important in that it defines the 'specific area' of the cement grind, that is, the total surface area of all particles per given mass of particles. The total specific area is the area that needs to be 'wetted' during mixing.

Different commercial sources of cement will possess a different particle size distribution curve, which supports the idea that the correct w/c ratio be established by laboratory testing of each source of cement used.

Transport and storage are known to affect specific area in that humid conditions and stacked loading of cement bags both reduce the specific area by aggregating the particles into tiny lumps.

In civil engineering, cement is stored in strict conditions and must not be used when its age exceeds 20 to 30 days. In comparison, in the mining industry, it is unlikely that the age of the cement is known by mine personnel.

4.4 Setting and hardening of cement paste

The setting and hardening of cement paste are two distinct phenomena associated with cement grouts but both are caused by chemical hydration reactions between the cement compounds and water. Setting refers to a state change from a fluid to a rigid paste whilst hardening refers to strength gain in the set paste. After setting, the paste volume remains constant but the internal structure changes as water and cement particles react to form products of hydration called gel. Hydration begins when water is mixed with cement powder to form a cement paste structure. As hydration continues, 'gel' and 'crystalline' products are formed that bind the paste into a coherent mass. The amount of gel and degree of crystallisation dictate the final strength of the grout. For most Portland Cement, initial set occurs in about 45 minutes but final set depends on the cement constituents and the temperature at which the hydration reactions occur.

4.5 The microstructure of cement pastes

The microstructure of the paste consists of a network of capillary pores and gel pores which affect porosity. The porosity of the paste affects its final strength and durability. The overall porosity of the paste is usually in the range 30% to 40% with the gel pores comprising about 25% of the porosity and the capillary network the balance (Jastrzebski 1976). The pores are gradually filled with hydration products as the paste sets and hardens. The rate of strength development gradually decreases with time and reaches a plateau region at about 28 days, which is a characteristic of most cement types.

4.6 Fluidity and workability of cement pastes

Fluidity is the ability of the grout to flow and be pumped which is very important in rock reinforcement applications. Workability is usually associated with concrete and the ease with which it flows with vibration, however, formally it is defined as the amount of internal work required to produce full compaction of the grout. This means the work required to overcome friction between the paste and any form boundaries (e.g. borehole wall) and the removal of entrapped air. In reinforcement practice, internal work is

restricted to that done by flow during placement. Consequently, the cement paste needs to be close to maximum compaction as it enters the borehole.

4.7 Bleeding and sedimentation of cement paste

Bleeding refers to the process of water seeping from capillaries in the cement paste. Bleeding changes the water cement ratio and may give rise to a heterogeneous column of grout with heterogeneous properties. Sedimentation refers to the gravitational settlement of any solids in a grout column, which usually results in decreasing grout density with grout column height, especially if a sand fraction is used.

4.8 Expansion, shrinkage and permeability of cement paste

Hardened cement pastes change volume according to changes in moisture content. Swelling and shrinking respectively accompany wetting and drying. In general, the water/cement ratio of the paste and the extent of curing dictate expansion, shrinkage and permeability. The grout's final resistance to water penetration and to chemical and corrosive attack by aggressive agents is all related to permeability. The less permeable the grout the greater the resistance of the grout. There is an exponential increase in permeability at a w/c ratio exceeding about 0.5.

4.9 Water

Water is the catalyst for cementitous grouts. The (w/c) ratio is given by the mass of free water relative to the mass of cement powder comprising the mix. The w/c ratio is known to affect most if not all of the chemical and mechanical properties of the fluid and hardened product. In the mixing and placement process it affects the heat generated, the rate and completeness of hydration, the fluidity, workability and bleed. In the hardened state it affects permeability, density, strength, stiffness, and creep of the grout. The theoretical range of effects of the w/c ratio on compressive strength, and stiffness presented by Thompson & Windsor (1999) are shown together with test data given by Hyett et al. (1992) in Figure 5. The expansion of the theoretical limits at low w/c ratios, confirmed by the scatter of experimental data is a critical observation when discussing the suitability of low w/c ratio grouts.

The ideal w/c ratio of cementitious grout for rock reinforcement is a compromise between the often-conflicting requirements of the grout in its fluid and hardened state. In order to achieve complete hydration of GP cement and thus optimum properties of the cured grout, a w/c of approximately 0.38 is often cited. Thompson & Windsor (1999) derived a theoretical value of 0.42 but its accuracy depends on specific area. Furthermore, to achieve adequate workability and a uniform distribution of water to wet and hydrate all the particles, w/c ratios well in excess of 0.38 are usually required in practice, (Shaw, 1980). But because the w/c affects shrinkage (c.a.0.5% at a w/c of 0.35 rising to 1.5% at a w/c of 0.5) and permeability (exponential increase in permeability at a w/c ratio exceeding about 0.5) it is essential to keep the w/c to a minimum consistent with the requirements of pumping the fluid.

The presence of moisture is required to complete the hydration process, known as curing. For a w/c of about 0.45, the amount of water available at the time of placement is usually sufficient for curing purposes. If the grout is kept continuously moist (i.e. in damp rock) it will cure in the borehole. Drying stops curing and prevents the significant strength gains required. This will be especially important in hot or dry rock that may extract and adsorb water and in fractured rock where the presence of discontinuities may also lead to capillary loss from the fluid grout.

The higher the w/c the greater the number of voids and capillaries that develop in the grout as the water bleeds to free surfaces. As the number increases, it becomes increasing difficult for the voids and capillaries to later fill with gel and become discontinuous, regardless of curing time. At a w/c of 0.6 it takes about 6 months of prefect curing to render the capillaries discontinuous. At a higher w/c this is impossible. Consequently, w/c ratios must be limited to below 0.45 to 0.5.

At the other end of the scale, w/c ratios of below 0.4 are sometimes used in ideal placement conditions with high quality mixing and pumping equipment. However, at w/c below 0.4 it becomes increasingly difficult to mix, pump and cure the grout properly. Figure 5 indicates both experimentally and theoretically that consistent high strength is not assured below a w/c ratio of about 0.4–0.42. In fact, depending on equipment and conditions of placement, attempting to use a low w/c ratio grout in pursuit of increased strength might result in host of more serious problems (e.g. high void ratios and higher permeability). The pursuit of extra strength in cementitious products by reducing the w/c ratio is a dangerous concept and the devastating effects are common knowledge in civil engineering practice. These include 'concrete cancer' and increased steel corrosion rates due to the inability to properly place the thick pastes without encapsulating air pockets.

It is reasoned on a technical basis that a w/c ratio of between 0.4 and 0.45 will provide the best chance of achieving a reasonable installation. In fact, the results of a worldwide survey reported by Weerashinghe (1993) indicated that the w/c ratios used for ground anchors range between 0.4 and 0.45.

Thick grouts at low w/c ratios can be properly mixed and placed but require proper equipment. However, their strength and permeability must be also be confirmed by testing the product cured in situ and not artificially in the laboratory.

Previous discussion indicates the conflicting requirements on w/c ratio of correct mixing and pumping with that of achieving optimal final properties. It is suggested that the final in-situ product be sampled and tested to define the in situ material properties actually achieved by the mixing and pumping machines used on the mix. In the light of published information on the high variability of grout properties this is a critical issue.

4.10 Water quality

In general, water with a pH of greater than 5.5 is recommended for grout used in rock reinforcement practice. FIP 1996 found that below this level the water is aggressive to hardened cement paste. However, such quality water may not be immediately available underground and consequently the water used must be checked for acidity/alkalinity by simple pH testing. This test may also point to the presence of contaminants (e.g. oxidation of pyrites and formation of sulphuric acid). Contaminants of most significance

here are chlorides, sulphates, carbonates and bicarbonates, all of which affect set and reduce final strength.

Furthermore, under certain conditions, chlorides initiate and accelerate corrosion of steel reinforcement and sulphates are responsible for sulphate attack of cementitious grouts.

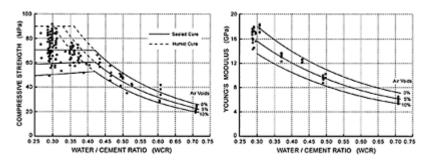


Figure 5. Variation in theoretical and experimental compressive strength and stiffness for different water/cement ratio grouts.

Table 3. Limits of dissolved contaminants in mix water.

Contaminant	Limit in water (mg/L)
Chlorides	500
Sulphates	800
Sulphides	100
Carbonates	2000
Bicarbonates	2000

Limits on common contaminants in water have been suggested by Xanthakos (1991): <0.5% chloride and <0.1% sulphate. FIP 1996 found in agressivity studies that 'hardness' or CaO content should be >30 mg/litre. Limits compiled from information given by the Cement and Concrete Association of Australia (C&CAA 1993) are shown in Table 3.

4.11 Grout admixtures

An admixture is defined here to mean an ingredient that produces a desirable change in the chemical and/or physical properties of the grout when added to the mix at some specific time. An admixture may be added prior to, or during mixing to produce a desirable result during mixing, pumping, setting or final hardening. Admixtures usually modify the properties of the paste by either:

- Introducing fine particles into the mix.
- Introducing air bubbles into the mix.
- Altering the rate of formation and/or the structure of the gel and crystallites.

The action of the admixture may be influenced by:

- The chemical composition and fineness of the cement.
- The chemical composition and temperature of the water.
- The timing of introduction into the mix.
- The amount of the dose.
- The mixing process and duration of mixing.
- The age and storage history of the admixture.
- The use of admixtures with other admixtures.
- The thermal and chemical environment in the borehole.

Admixtures like all components of grout must comply with a set of specifications that regulate their composition and compatibility and aim to ensure their chemical and physical effect on the grout.

Australian Standard AS 1478—Chemical Admixtures for Concrete (ASA 1992) define the compliance requirements for a number of generic admixture types, including the first 8 listed in Table 4. Similarly, the final three admixtures listed in Table 4 must comply with Australian Standards MP20 Part 1 (Permeability Reducing), MP20 Part 2 (Thickening) and MP Part 3 (Expanding) (ASA 1975) respectively.

Admixture	Туре
Air Entraining	AEA
Set-Retarding	Re
Set-Accelerating	Ac
Water Reducing	WR
Water Reducing and Set-Retarding	WRRe
Water Reducing/Set-Accelerating	WRAc
High Range Water Reducing	HWR
High Range Water Reducing and Set-Retarding	HWRRe
Permeability Reducing	PRA
Thickening	ТА
Expanding	EA

Table 4. Admixture naming convention.

Air Entraining Admixtures produce a system of minute air bubbles dispersed throughout the mix. This is useful in cold environments where they greatly improve resistance to freezing and thawing effects. AEAs also improve cohesion, plasticity, and workability and affect durability, bleeding, permeability and shrinkage. However, AEAs decreases final strength as a proportion of the amount of air entrained. Consequently, AS 1478 limits their use to about 6%, which ensures that, the final grout strength is about 90% of that without the admixture.

Set-Retarding Admixtures increase the setting time from a fraction of an hour to a few hours, which is particularly useful in hot environments where pumpability needs to be maintained. This is not a significant problem in normal conditions but may be important around deep, oxidising ore bodies where rock temperatures may exceed 45°C.

Set-Accelerating Admixtures reduce the setting time and are useful when high early strength development is required. One popular accelerator is Calcium Chloride, the side effects include increased creep and promotion of corrosion of steel reinforcement.

Water Reducing Admixtures disperse the cement particles uniformly throughout the mix to increase its fluidity. This means they can be used to decrease w/c and increase strength while maintaining workability. However, one side effect is to delay hydration and thus retard set time. This may be compensated by including an accelerating compound in WRs. This compensation gives rise to the three chemical variations (i.e. WR, WRRe and WRAc), where each is formulated with different objectives in mind.

The High Range Water Reducing Admixtures, HWR and HWRRe allow normal and retarded set times respectively and are often called 'super plasticisers'. In fact, they increase fluidity not plasticity and may be used to significantly increase fluidity while maintaining a given w/c ratio and strength requirement.

Permeability Reducing Admixture requirements are specified in SAA MP20 Part 1. These are formally recognised as being able to reduce, as opposed to prevent, the transmission rate of moisture through the grout. Unfortunately, in some literature they are termed 'waterproofing' agents but their effectiveness is variable. Some are known to have detrimental effects on strength, shrinkage and workability.

Thickening Admixture requirements are specified in SAA MP20 Part 2. They aim to increase viscosity of the mix and can also reduce bleeding and improve the grout bond to steel reinforcement. They are subdivided into 4 classes (A, B, C and D) depending on how they affect the interaction of the cement particles during mixing.

Expanding Admixture requirements are specified in SAA MP20 Part 3. They provide expansion in the mix in the plastic or hardened state by either expanding themselves or reacting with other compounds to cause expansion. They are particularly useful in reducing net shrinkage and will expand in the borehole. If the fluid grout column in the borehole is constrained longitudinally (and radially) then the expansion may be sufficient to marginally increase the grout/reinforcement interface confinement. These admixtures are subdivided into 4 classes (A, B, C and D) according to their physical action on the grout. A commonly used expanding admixture is aluminium powder, which unfortunately generates hydrogen gas bubbles in GP cement mixes. This is not recommended for use with high tensile steel reinforcements not only because of the risk of hydrogen embrittlement but because it reduces the fatigue resistance of steel and increases grout permeability.

This brief review of admixtures highlights a number of points concerning their use:

- Behaviour is dependant on numerous factors.
- May provide desirable and undesirable effects.
- May interact and affect one another.

Development of admixtures is an ongoing science (Pearse 1994) and their use requires strict adherence to formal recommendations. Here, it is suggested that they should not be used in rock reinforcement practice either individually or collectively until the mechanical and chemical properties of the grout produced from the proposed mix design have been researched, tested and studied in detail. Furthermore, once a mix design has been approved, it is important that procedures for ensuring quantity metering, mixing and pumping are standardised and that training and supervision are increased to avoid mistakes in practice.

Clearly, a technologically advanced mix for say, managing a specific site problem requires a advanced work practice and trained staff to implement it. However, an important point, confirmed by worldwide ground anchor practice, is summarised by Bruce (1993) '...neat grouts, when properly mixed and placed are nearly always adequate'.

5 THE EXTERNAL FIXTURE

The external fixture of most cable bolt devices involves an assembly of steel plate, steel barrel and wedge fittings (or grips) and if necessary, bearing washers and seats. These components are arranged to fix and pre-tension a decoupled, short length of strand at its proximal end to the rock surface. Here we will explore the plate and grip requirements prior to discussing the process of pre-tensioning the element and properly locking that tension into the strand with the external fixture.

5.1 Plates

There are three basic requirements of the plate. Firstly, to avoid increasing the corrosion potential of the installation, material types must be similar, thus galvanised steel plates should be used with galvanised strand and plain steel plates with plain strand. Secondly, the plates require a number of centrally positioned holes or slots of slightly larger size than the strand diameter. In multiple strand installations the holes should have a minimum spacing defined by the grip diameter and a maximum spacing defined by the grip diameter and thickness should be dimensioned according to the ultimate strength of the strand(s) in terms of both plate flexure and pull-through of the grips through the holes.

In single strand installations, one central hole is sufficient but installation and action may be improved by using an elongated slot. This is especially true when the plate is not perpendicular to the strand axis in which case, the slot provides a limited opportunity to rotate the strand axis. In twin strand installations, two holes are required, spaced sufficiently apart to allow proper seating of side-by side grips but not too far apart to cause bending of the strand as it exits the borehole collar and passes through the plate hole. Again, two elongated slots, especially when small diameter boreholes are used or when the plate is not perpendicular to the strand axis facilitates this.

The best way of coping with single and multiple strands that, due to borehole orientation not being perpendicular to the plate, is to use grips with hemispherical bases and matching seats (plate slots assist only marginally in this circumstance).

The practice used for twin strand cable bolts of fixing only one strand is not considered good economic practice. The practice of bending one strand under the plate and fixing the other is considered a practice that should be avoided especially in the case of securing mine infrastructure excavations.

The use of slots instead of holes in plates and the use of hemispherical grips instead of plain grips increases the cost of the installed cable bolt system slightly. The advantage is a system that can utilise the capacities of all the components to perform the task of securing the rock properly.

5.2 Grips or barrel and wedge anchors

The rationalisation of wire and strand materials in the prestressed concrete industry has resulted in standard compliance requirements that ensure high axial performance of the reinforcement element. However, there are no known engineering standards for fixture at the element terminations. Fortunately, in both the prestressed concrete and ground anchor industries the installed system (which by definition includes the external fixture) is required to pass stringent prestressing, relaxation and creep proof testing standards in situ.

These industries have developed fixing devices termed grips and grip installation procedures that satisfy the rigorous proof testing requirements. In mining rock reinforcement practice, fixture designs and procedures have simply been copied from these disciplines on the understanding that they have been proven suitable. This remains a valid assumption as long as the design of the fixture and the process of installing it are faithfully copied. Unfortunately, in some instances this is not the case.

5.3 Grip designs

Some of the grip styles used in the mining industry today are shown in Figure 6 and comprise the flatended universal grip, the hemispherical ended grip and the conical ended grip. The first two are used as external fixtures in conjunction with plates to fix the strand to the rock surface. The third may be used as an internal fixture in the grout near the distal end of the strand. This grip may be used to control displacement in a system that has been provided with a yielding interface between the grout and the strand (e.g. sheathed, coated, painted or plain strand.)

The early fixtures for prestressing elements were designed in Europe about 100 years ago by Freyssinet for use on plain wires in bridge construction and comprised various arrangements of collets and wedges. By the early 1940's, 7-wire strand become the universal standard prestressing element in Europe and the 3-component wedge inside a flat-ended barrel became the standard grip design (Burgoyne 2003). The CCL prestressing grip introduced in 1951 (CCL Pty Ltd 2004a) using 3 component, precision wedges with high quality, dimensionally matched steel barrels is an example of the universal grip design currently used in civil engineering.

Subsequent modifications have led to a range of grips based on this generic design which include:

- The Standard Open Grip.
- The Spring Loaded Threaded Cap Grip.
- The Spring Loaded Bayonet Grip.

The three variants suitable for nominal, 15.2 mm strand all comprise a 50 mm diameter steel barrel with a three-component wedge. They have a common load capacity of 300 kN which exceeds the axial load capacity of the strand. The three-component wedge is used on strand to ensure a better fit around the circumference of its non-prismatic section. All three variants include an 'O' ring or circlip that controls wedge alignment and prevents 'stepping' of the wedges.

The Standard Open Grip comprises an open-ended, 57 mm long barrel. The Spring Loaded Threaded Cap and Spring Loaded Bayonet Grips are similar but include a steel spring that bears on the wedge outstand surface and a threaded or bayonet steel cap to hold the spring in position. To accommodate the spring and cap in these two variants, the barrel is longer (103 mm). The spring-loaded versions are designed to press on and keep the wedges tight inside the barrel. Later discussion will reveal the importance of maintaining the position of the wedges.

The process of fitting a grip to an element involves stretching the element while pushing the barrel against the bearing plate surface. The wedges are then placed within the barrel around the element and the element is then released, pulling the wedges further into the conical recess and forcing the wedge teeth to bite into and clamp onto, the element surface. Note, that it is the elastic rebound of the stretched strand that sets the wedges when it is released and not the jack. The dimensions and tolerances of the wedge curvature, the conical recess in the barrel and the strand diameter are critical for ensuring correct 'bedding-in' and 'pull-in' of the wedges. Correctly designed grip sets are dimensioned for specific barrel and strand dimensions and should not be mixed. A universal grip with correctly set wedges is shown in Figure 7.

The original grips used in mining rock reinforcement were the Standard Open Grip for external fixtures, a back-to-back, double variant for double acting intermittent internal fixtures and a reduced diameter, two-component wedge copy. The nonstandard, reduced diameter, two-component wedge copy was developed specially for internal fixtures at the distal end of the element and not for the external fixture (see Section 5.6). Currently, the most popular grip used as an external fixture in cable bolts is a twocomponent wedge copy of the original open grip. Unfortunately, a proportion of the grip copies used in mining are not sufficiently accurate copies to enable them to function properly.

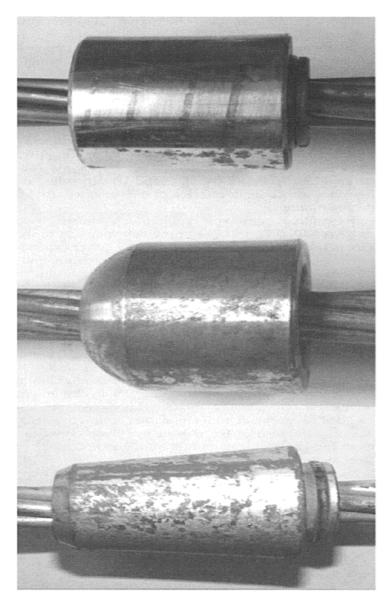


Figure 6. Three different grip designs for three different applications. Top: A flat ended universal grip, Centre: A hemispherical ended grip and Bottom: A conical ended grip.

5.4 The effects of variation in strand diameter

Because standards associated with prestressing elements are concerned mainly with axial performance, the tolerance on nominal element diameter is not

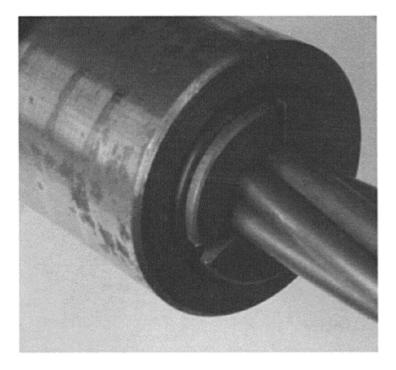


Figure 7. A universal 3-wedge grip with correctly set wedges.

considered overly important. For example, AS 1311, (SAA 1978) states:

'The tolerance permitted on the strand diameter, measured across the crowns of the wires, shall be ± 0.4 mm'.

This means that strand with a nominal diameter of 15.2 mm may range from 14.8 mm to 15.6 mm diameter. This is acceptable in regulated industries were any dimensional variations that effect system behaviour are discovered during proof testing.

However, the author has measured non-compliant strand diameters in excess of these tolerances. This is an important observation which has significant consequences for grip performance and for pretensioning rock reinforcement. An oversize strand will increase the wedge 'outstand' where an undersize strand will increase wedge 'recess'. Both are shown in Figure 8 and both limit the pretension that can be locked into the strand. In civil engineering this is accounted for by correctly matching the grips to the strand batch and then proof testing the installation. Examples of excess wedge outstand or excess wedge recess that have been observed in mining and indicate that the internal dimensions of the barrel and or the wedges are incompatible with the diameter of the strand used.

5.5 Two and three component wedges

Earlier discussion indicated that originally the 2-component wedge was designed for wire and the 3-component wedge was designed for strand. It was found in research conducted at CSIRO that a correctly designed and correctly installed two-component wedge grip applied to nominal 15.2 mm diameter strand is almost compatible with the ultimate axial load of the strand. Our tests showed that failure commonly initiated at the two-component wedge fixture at or only slightly below the strand axial force capacity. The initiation of strand failure due to stress concentration at the wedge teeth can significantly reduce the axial elongation capacity of the system without significantly reducing axial load capacity (viz. loaddisplacement curve for strand). Most mining applications of strand cable bolts use two-component wedge grips.

If there is compliance with all the other design rules for grips then a change from the recommended 3-component wedge for strand to the 2-component wedge is acceptable. That is, given that any reduction in axial strain capacity of the system, which may be imposed by the grip (obtained by testing), is acceptable for the collapse mechanism being reinforced.

After proper setting, the 2 wedge component grips often show a radial cracking of the wedges due to mismatch of the wedge/barrel surface under strand load (e.g. Figure 9). Radially cracking of the wedges does not seem to greatly affect performance whereas any transverse cracking caused by preassembly damage or during inappropriate setting would.

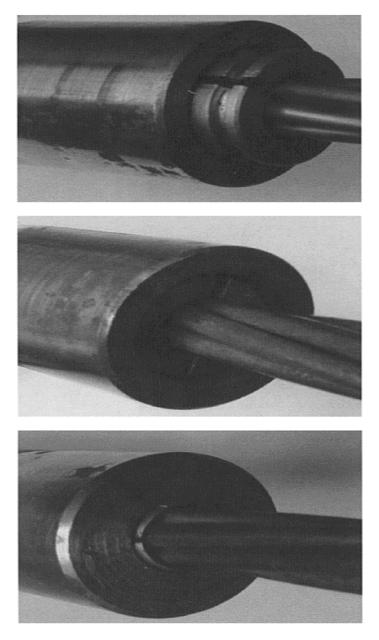


Figure 8. Two wedges grips. Upper: Wedge outstand. Centre: Wedge recess. Lower: Protrusion of the wedge tips.

5.6 The effects of barrel diameter, steel grade and friction

The properties of the steel used and the external dimensions of the barrel are also important. Barrels dimensions must exceed the minimum dimensions (for a given steel grade) to prevent 'splitting' or 'bursting' of the barrel under internal radial load from the wedges.

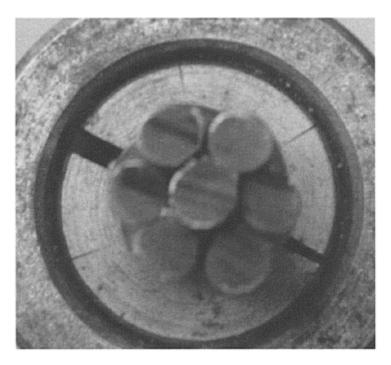


Figure 9. Radial cracking of wedges (i.e. at 40°, at 260° and at 355° around the circumference) in a two-wedge grip.

Research and testing of barrel dimensions was conducted at CSIRO in the early 1980's to define the minimum barrel dimensions for a given steel quality for barrel and wedge grips installed as intermittent anchors within the grout. Intermittent anchors may be designed to be yielding or non-yielding although all internal grips cause yielding of the grout during pullout of the strand. They may be used in conjunction with a yielding strand/grout interface where the grip is designed to control system stiffness and ultimately, system displacement. The behavior of one yielding grip design is shown in Figure 10.

Special non-yielding, barrels for use as internal fixtures were produced by Rock Engineering Pty Ltd and were designed to minimise borehole congestion. These designs took into account that, in service, they had the significant advantage of radial confinement provided by the grout and the rock.

In the case of an unconfined external fixture, the standard barrel dimensions must be maintained—the two cases should not be confused. For example, the CCL Standard Open Grip barrel has an external diameter of 50 mm, an internal wedge taper angle of approximately 7 degrees and a capacity of 300 kN. However, given that a high-grade steel is used, a 45 mm barrel is also acceptable. In fact this dimension is preferred for multiple strands in production boreholes as its enables the strands to be fixed with side-by-side barrels.

The behaviour of both 50 mm and 45 mm and diameter variants have been presented by Marceau et al 2001, 2003. They give results from numerical and experimental studies into the relationships between the barrel dimensions, the wedge taper angle, the friction between the wedge and the barrel and the effects on the stress distribution in the barrel at elevated service loads for '3-component' wedges. This is recommended reading for grip manufacturers. However, within that work the authors state that:

"...the change of angle of the wedge outer surface and lubrication of the wedge-anchor head interface can lead to poor performance of the mechanism; collapse of the anchor head could happen".

This observation is valid for radially unconfined grips in the context of the barrel bursting mechanism. But, great care is required in interpretation of this comment with general regard to lubrication of grips. It is certainly true, that a lubricated wedge/barrel surface will increase 'draw-in' of the wedges and thus circumferential tension in the barrel, which in the case of an incorrectly designed barrel may lead to splitting or bursting. However, for the preferred case of a correctly designed barrel, lubrication is not a problem but a fundamental requirement to ensure correct operation of the grip.

Over the last decade, Thompson has repeatedly shown and warned of the importance of reducing friction at the wedge barrel interface (Thompson 1992—Thompson 2004). In fact, without smooth, clean, lubricated surfaces it is impossible to set the grip properly and it will be found that the grip will pull off the strand at loads substantially less (e.g. 50%)

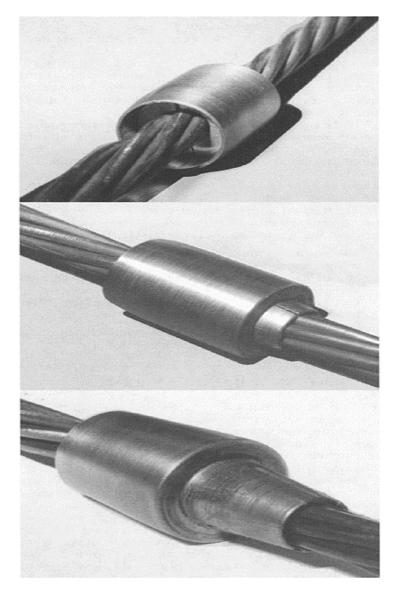


Figure 10. The behaviour of an unconfined, yielding internal anchor during wedge 'pull through'. When placed in grout the grip may be used to control the stiffness and displacement of the system once the strand/grout interface has yielded. than the capacity of the element. Many barrels now used in the mining industry have a rapidly machined, relatively rough internal surface and very few if any are lubricated. The apparent cost advantage is incompatible with the potential loss of performance.

Clearly, if barrels are galvanised, the internal slide surface will be 'rougher' due to the coating and should be re-machined and lubricated.

To comply with Thompson's recommendations, barrel and wedge grips are required to be free of corrosion and dirt and need to be lubricated. The grips and particularly the brittle wedges should also be examined for any cracking prior to assembly and should never, under any circumstances, be subject to direct impact (CCL Pty Ltd 2004b).

In the past, undersized barrels, incorrectly matched barrel and wedge sets and barrels made from substandard steel have been imported and or manufactured in Australia. A detailed analysis of the effects of geometry, friction and corrosion on grip behaviour together with field and laboratory proof is given by Thompson, 2004.

The factual and anecdotal evidence suggests that manufacturers/suppliers/users of barrel and wedge grips in the Australian mining consider three issues:

- The geometry and steel grade of the barrel and wedge assembly.

– The behaviour of the assembly during installation and prestressing.

- The behaviour of the assembly in service, after lock-off, at elevated service loads.

The mining industry should purchase these components on the basis of performance not cost.

6 PRE-TENSIONING THE SYSTEM

The pre-tensioning elements is the most important process in both ground anchor and prestressed concrete industries where strict procedures exist for each step of the process. Tensioning of cable bolts has been researched and described by Thompson (1992), Thompson & Windsor (1995) and Thompson (2004).

The initial method of pre-tensioning elements was borrowed from civil engineering practice with modification to the process and the equipment in order to pretension a relatively short length of element near the collar. This was undertaken to ensure that the high strength, high stiffness, plates introduced were installed tight against the rock surface. Prior to the use of plates, the rock surface simply fretted away. Prior to pre-tensioning, the stiff plates were invariably loose which meant that the rock face had to deform a significant amount to achieve a response from the element.

Laboratory and field tests have shown that if the recommended pre-tensioning equipment and procedures are used, plates can be installed properly with a prestress locked into the strand.

In the initial attempts at pre-tensioning to pull the plates in tight against the rock, a short length of element was decoupled from the grout using a plastic tube or a spiral wrap of plastic tape. The initial prestressing jack designs were borrowed from prestressed concrete technology and went through a development process of some years (CSIRO/Rock Engineering Pty Ltd, Confidential Research Projects, 1980–1985) involving weight reduction and a sequence of modifications to the nose cone. The

sequence of design modifications to the prestressing jack nose cones is given by Thompson & Windsor (1995) and are shown schematically in Figure 11.

The first, flat, nose cone design (Method 2) applied force directly on the wedges. In research conducted at Mount Isa in 1984, the author found by instrumentation of the free length, that the load registered by the jack was not being locked into the element. The second, stepped, nose cone design beared directly on the rim of the barrel before engaging with the wedges at a predetermined outstand and was found to be an improvement. The third, spring loaded nose cone design (Method 3), beared directly on the rim of the barrel and the spring pushed the wedges into position with a given spring force. Excellent results were

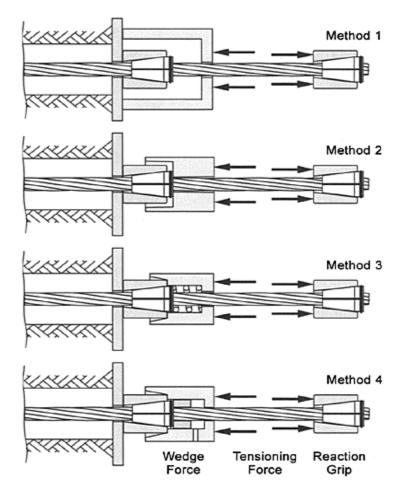


Figure 11. The evolution of cable bolt stressing jacks.

obtained with this method, especially on sites with sufficient discipline to replace the spring as it softened with use and age. The ultimate nose cone design, and the one used in civil rock reinforcement involves a small, secondary hydraulic ram that pushes on the wedges with a constant force (Method 4) once a set load had been applied to the element by the primary ram. This completely hydraulic arrangement achieves proper setting of wedges with automatic lock-off at the required pretension in the element.

The installation procedure used on a number of Australian mine sites was found to result in either no pretension being applied to the strand, damage to the wedges or damage to the strand. One method involved setting the wedges by a steel hollow tube hammer, with impact blows being applied direct to the wedges. This very quick, very simple, procedure leads quickly to wedge damage and simply to no pretension being applied to the element. Other methods have been observed that damage the strand and render the fixture unable to operate under load. For example, pushing on the wedges to set them in the barrel causes the sharp wedge teeth to plane off a layer of steel from the strand wires filling the wedge teeth fill with steel filings. This firstly prevents them from embedding in the strand properly and secondly provides a smooth planned surface on the strand enabling the grip to slide off under load, especially under dynamic load. Note that potential sliding grips can be identified by inspecting the installation for signs of strand damage leading into the back of the grip.

An advantage of the spring-loaded grips is that they hold the wedges in position. These are used as a 'dead anchor' at the distal end of a strand in a prestressed concrete member. The dead anchor may be partially set by hand at the distal end and then automatically set as load is applied at the proximal end. In a cable bolt, if the wedges can be partially set and held in position without damage to the fixture then the strand will complete proper seating of the wedges in the barrel during initial loading from the rock. Unfortunately, this means that no pre-stress is established and the initial axial stiffness of the system is lower than that with a properly installed grip. This is not recommended practice, however it is considered preferable to a procedure where the grip has either been destroyed, rendered inoperable or may slide during service due to strand damage.

7 CORROSION OF THE SYSTEM

Before discussing corrosion it is useful to consider a recommendation from AS 1311 (SAA 1987), which states:

'Light surface rusting is permissible, provided that no pitting is visible to the naked eye'.

Virtually identical comments may be found in most national standards for it has long been recognised that light rusting does not alter the mechanical properties of the element. In early work, Fuller and Cox (1975) found from laboratory testing on plain prestressing wires that light rusting, marginally increases the apparent bond strength by increasing the coefficient of friction at the wire/grout interface. These findings have recently been confirmed for strand by Lopes and Simoes (1999) who found that surface rusting does not overly effect any of the important mechanical properties of strand but does increase the coefficient of friction at the strand/grout interface.

Although, both sets of conclusions are valid, the practical interpretation of these findings have caused problems in the past where light surface rusting has been confused underground with more serious conditions (e.g., a hardened coat of oxidising slimes). Furthermore, it is known from tests on 'indented' strand that pitting will marginally increase mechanical interlock and thus bond strength at the interface. However, if pitting is present then the strand is actually corroding and the pitting may become a site of stress concentration. Consequently, since the early 1980's it has been recommended that the strand be clean and free from rust and pitting.

All reinforcing elements produced from steel are susceptible to corrosion from their in situ environment. For example, The Berlin Congress Hall collapsed in 1980 (Beton und Stahlbetonbau 1980) due to corrosion of prestressing strand tendons in the above ground roof. When steel elements are placed within the ground the risk of corrosion is greater due to the presence of ground waters that may include chlorides, acidic compounds and other aggressive components. When the ground includes the oxidising ores and minerals commonly found in metalliferous regions the corrosion potential is higher still.

Corrosion is a thermo-electro-chemical process that occurs between cathodic and anodic regions that may occur around a void or a crack that extends through the grout to the reinforcement. The anode is set up at the void where oxidation of the steel occurs with electrons being lost and consumed by a cathodic region either side of the void. The steel acts as a conductor between these regions and the pore fluid acts as an electrolyte. Corrosion reduces the cross sectional area of the steel (and thus increases stress) until the anode is exhausted of electrons which may eventually lead to failure.

Cementitous grouts based on Portland cement possess high alkalinity (e.g. pH typically in the range 12 to 13). However, hardened cementitous grout reacts with carbon dioxide (carbonation), which reduces the pH and allows the onset of corrosion, especially if the oxide film that develops at the surface of the steel is breached. Hardened cementitious grout is also susceptible to Chlorides, which can penetrate the grout to depths of over 50 mm and initiate the corrosion process (see Rasheeduzzafar et al. 1992).

Elements that are pre-tensioned, or become loaded to reasonable high loads are particularly at risk. In this case, not only is the stress increased with loss of cross sectional area but the stress can accelerate corrosion in what is known as stress corrosion. It been known for some time that nitrates, hydrogen and hydrogen sulphide aid stress corrosion. The results of a recent study on stress corrosion of prestressing steel have been given by FIB (2003).

The corrosion rates of rock reinforcement in service are extremely difficult to define due to the difficulty of defining the 'corrosion potential' of the chemical compounds surrounding the anode and of those comprising the electrolyte. In response to concerns about the corrosion of prestressed strand in approximately 100,000 bridges in the USA, the National Cooperative Highway Research Program (NCHRP) extensively researched the capability of Non Destructive Test methods to determine the in situ condition of embedded prestressing strand and reported that no method existed that could satisfactory define the condition of the strand. This work has been reported by Ciolko and Tabatabai (1999). Only one technique was found to hold future promise, the magnetic flux leakage method, but that method required considerable development. In conclusion, it is extremely difficult to define:

- The in situ 'corrosion potential' of an environment.

- The in situ 'condition' of embedded prestressing strand.

A study on the corrosion potential of the environment and the corrosion condition of various reinforcement devices was conducted at the Mt Isa mine from 1995 onwards and has been described by Robinson and Tyler (1999). This study provides a review of the corrosion of reinforcement and also sets out how a mine might conduct a site-specific corrosion study.

The preliminary results from research currently being conducted at WASM, Curtin University to study corrosion of steel reinforcement in mining environments has been described and discussed by Hassell et al. (2004) in terms of the thermo-electro-chemical environment. The results from this work will hopefully indicate the corrosion potential of certain mine environments and the corrosion rates of reinforcement in those environments.

Once this data has been obtained decisions will be required to be made on suitable measures for protecting the reinforcement used for non-temporary, infrastructure excavations.

7.1 Corrosion protection standards

The most extensive worldwide study on prestressing elements was conducted by the Federation Internationale de la Precontrainte (FIB 1986) during the preparation of the corrosion protection standards for ground anchors. Of the millions of ground anchors installed since the first systems were used in the Cheurfas Dam in 1934 (Littlejohn 1992), the FIP study found 35 incidents of failure by corrosion. This very low frequency of failure in below ground, reinforcement systems in civil engineering is undoubtedly due to careful installation practice and using physical barriers to surround the element to prevent the ingress of the electrolyte. However, this figure can be expected to increase as devices installed prior to the standardisation of protection measures are found to be faulty as they progressively age and deteriorate. The FIP study found that corrosion is usually localised within the reinforcement length and that the external fixtures are particularly vulnerable, (Barley and Windsor 2000). These findings have been confirmed by later studies.

A discussion on the corrosion protection measures in the 1996 European Standard, EN1547 has been given by Merrified et al. (1997). EN1547 states:

'There is no certain way of identifying corrosion circumstances with sufficient precision to predict corrosion rates of steel in the ground. All steel components which are stressed shall be protected for their design life'.

For permanent installations it recommends a single layer of corrosion preventative material provided that in situ testing proves the integrity of each device installed. Installations not tested in situ must be supplemented with two protective barriers and the competence of this double layer system must be confirmed by at least one system test. EN1547 provides tabulated details of suitable protection systems for both temporary and permanent installations.

Although Australia is at the forefront in the civil engineering application of long (up to 120 m) ultra high capacity (up to 12,000 kN) installations complete with modern

corrosion protection systems (e.g. Cavill 1997) there are no standards for corrosion protection of cable bolts. Furthermore, it will be suggested later that some cable bolt installation practices may actually provide conditions that assist corrosion.

7.2 Corrosion protection of steel strand

The various options for protecting steel reinforcing elements include:

- Painting.
- Galvanising.
- Epoxy Coating and Encapsulation.
- Combination Coating.
- Sheathing.
- Combination Sheathing.

Painting the strand surface with Zinc rich or polymeric paints can increase corrosion resistivity but little is known of the relative cost benefit and performance compared to other procedures.

Galvanising involves coating a steel surface with Zinc with a minimum thickness of about 85 microns. It is known that galvanising increases resistance to chloride but it is not clear what levels of chloride can be tolerated without damage. Long-term trials in aggressive environments suggest it is not a panacea for all corrosion problems (e.g. Heiman 1986).

Further, if a cementitous grout is used and contains <65 ppm Chromium compounds, then hydrogen gas may be liberated in a reaction at the cement/zinc surface. Interestingly, Satola and Hakala (2001) found gas bubbles at the grout/steel interface over the complete grouted length of galvanised steel strand and stated that this seemed to increase the bond strength. Galvanising increases the strand surface roughness, which may increase mechanical interlock leading to a marginal increase in bond strength. However, gas bubbles result in a weak, spongy interface with a higher permeability, which is to be avoided. Consequently, in reinforced concrete practice the addition of 0.3 g of Chromium Trioxide per litre of grout mixing water used to be recommended when using galvanised elements in concrete when preliminary tests indicate the formation of gas bubbles at the interface. The use of galvanised materials is increasing and there are new material compliance codes to cover this development (e.g. The Draft Revision of AS 2841–1986 by AS 2004).

An epoxy coating, particularly epoxy fusing coating can, in some environments, provide an enhanced degree of protection from corrosion. The addition of silica or quartz particles in the fusion process results in a rougher surface of the coating with enhanced mechanical interlock with the grout. Epoxy encapsulation involves not only coating the outside of the strand but also filling the strand interstices to essentially encapsulate the strand wires. Epoxy protected strand has become increasingly popular in ground anchors for general application and for holding down gravity dams. ASTM 2001 has provided a standard specification for epoxy-coated strand and a historical review of developments has been given Bruce (2002) for the International Association of Foundation Drilling. A summary of the findings from this review has been given by Bruce and Barley (2003) together with recommendations for future use that will remedy some of the problems

identified in the early use of epoxy protected strand. Epoxy encapsulated strand is now a real candidate for corrosion protection of cable bolts, given that the recommendations on fixture are followed.

Combination coating involves the application of two different coating techniques. For example, Ostra Stalindustri (1991) have developed a two stage process involving an initial hot-dip, 80 micron galvanising layer followed by a thermo-hardened plastic coating of an additional 80 microns. In salt, fog tests over one year, unprotected steel specimens suffered a 50% reduction in thickness whereas the combinationcoated samples suffered no damage at all.

Sheathing the element is a highly effective treatment when slip is required over a certain interface length. The typical application is when a 'free length' is required to be decoupled from the grout such that it may be pretensioned. The strand is surrounded by an outer flexible (usually HDPE) sheath which is pumped full of wax or grease. Excellent results have been achieved in civil engineering practice where it is one of a number of recommended methods for improving corrosion protection. Sheathed strand is a suitable candidate given that the system provides the required stiffness and displacements are controlled by an internal anchor.

Combination sheathing involves the introduction of multiple layers to protect the element. The development history and design principles for using multiple layered sheaths for ground anchors has been given by Barley (1997). The simplest example is multiple plastic sheaths between the element surface and the borehole wall with the consecutive intermediate and outer annuli being filled with grout. The sheaths are corrugated in longitudinal section to effect a mechanical key between the consecutive cylinders of grout. If the element is to remain unbonded then a grease filled sheath is used on the element within a larger corrugated sheath, which is grouted internally and encapsulated by an outer layer of grout. Clearly, the greater number of layers the greater the protection.

In the end, the problem of corrosion protection of steel reinforcement remains one of:

- Defining the required service life of the reinforcement system.
- Defining the corrosion potential of the environment.
- Selecting a candidate protection system.
- Testing the candidate protection system at the corrosion potential of the environment.

7.3 Aspects of installation on steel strand corrosion

Some aspects of reinforcement installation may introduce pre-service faults and defects into the reinforcement system that may initiate and accelerate the process of corrosion. These include:

- The water-cement ratio of the grout.
- The degree of encapsulation of the element.
- The fault/defect orientation and its dimensions.

Previous discussion indicated how the water cement ratio of cementitious grouts affects many of its mechanical properties. For example, permeability of the final grout is affected and a high permeability grout is more susceptible to penetration by an electrolyte or other undesirable chemicals than a low permeability grout. The degree of encapsulation affects the corrosion potential of an installation. For example, a properly spaced and centralised twin strand installation ensures that the strands are spaced sufficiently apart (minimum of about 5 mm) and held off the borehole wall. Given that a suitable grout mix and grouting procedure is used, the strands can be fully encapsulated along their length. When spacers and centralisers are not used, the strands may contact each other and the borehole wall along their length (especially true in the case of non-vertical boreholes). This interference prevents full encapsulation of the strand and reduces axial capacity of the cable bolt. Furthermore, depending on the grout mix and/or the grouting procedure 'bubble' and 'tunnel' shaped air voids may be introduced into the grout column which later, may become the site of an anode complete with electrolyte reservoir.

The orientation and dimensions of an imperfection in the grout column surrounding an element affects the size of the anode and thus the rate of corrosion. Longitudinal faults caused by installation can be more severe than common transverse cracks. In this respect it is also important to recognise the consequences of the composite nature of strand. The strand cross section includes 6 helical interstices around the king wire. During the grouting of up-holes, water may bleed under pressure from the head of grout into the interstices and flow down to the collar where a constant stream of drips may be observed from the strand (Note, that this does not occur in down-holes). If the interstices eventually fill by pressure filtration with the products of cement hydration, then initial water loss is the only concern. If not, a longitudinal air void is present around the king wire. This problem is avoided in the case of encapsulated strand where the interstices are filled with epoxy. Similarly, an advantage of the modified strand geometries is that the path length of this longitudinal void is effectively broken at regular intervals along the strand axis (e.g., Birdcage or Bulbed strand).

When corrosion protection is required it should involve not only additional protective measures but also adherence to certain installation practices that minimise the chance of introducing circumstances which could assist corrosion.

8 CONCLUSIONS

This discussion has explored some issues that affect the quality and performance of the cable bolts used in mining engineering. The findings are not new; most have been recognised as problematic for some time by rock reinforcement research workers. Similar issues and concerns have long been noted in both prestressed concrete and ground anchor engineering. In both of the latter disciplines, the problem of ensuring quality control of reinforcement components has been accounted for by material compliance standards and the problem of assembled system performance has been accounted for by design codes and work practice standards. Furthermore, in these disciplines correct implementation is finally checked by the requirement to conduct final proof testing of the complete system. However, no such safeguards are formally required in mining engineering.

The quality and performance of cable bolts used to stabilise temporary, non-entry, production excavations have improved over the last 20 years to the point where they are now an essential part of modern mining practice. Cable bolts have provided the industry with increased production, increased safety and increased flexibility in the extraction

process. However, with the development of wider span haulage and other larger excavations, cable bolts are now also used to secure longer life mine infrastructure excavations. It is recommended that greater care and attention to detail be invested during selection and installation of cable bolts for mine infrastructure excavations than that given to mine production excavations. The details of significance include:

- The Element.

- Compliance with standards.
- Test Certification.

- The Internal Fixture.

- Fresh, properly stored cement.
- Clean uncontaminated water.
- W/c ratio established by testing.
- Admixtures used with caution.
- Suitable mixing equipment.
- Suitable pumping equipment.
- Test w/c on change of cement supply.

- The External Fixture.

- Matched material types.
- Correctly dimensioned plates.
- Allowance for miss-alignment.
- Correctly dimensioned grips.
- Matching wedge sets.
- Machined, clean lubricated barrels.
- Strand compliant with standards.

- Prestressing the system.

- Strand compliant with standards.
- Correctly dimensioned grips.
- Matching wedge sets.
- Machined, clean lubricated barrels.
- Suitable pre-stressing jack.
- Correct pre-stressing process.

- Corrosion of the system.

- Minimal voids.
- Materials compliant with standards.
- Low permeability grout.
- High chemically resistant grout.
- Corrosion protection method.
- Suitable installation procedure.

Securing mine production excavations with cable bolts requires our best effort. Securing mine infrastructure excavations with cable bolts demands it.

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1 Case studies

The evolution of ground support practices at Mount Isa Mines

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ABSTRACT: Mining has been in progress for over 80 years in Mount Isa. During this time the need to improve safety, productivity and economics has resulted in changes to underground mining methods and equipment. This has led to larger excavations, larger equipment and the development of new technology. This paper describes the changes in ground support practices that have occurred over the years to accommodate the need for improved productivity and safety. It will discuss the changes in ground support practices from installing no support to the introduction of rockbolting and meshing in each new development round; the introduction of cable bolting to different mining methods including cut and fill, benching and sub level open stoping and will describe current practices at Mount Isa Mines.

1 INTRODUCTION

Mount Isa Mines has been mining base metals in northwest Queensland for over 80 years. This has been predominantly by underground mining methods in two broad distinct ore types, copper and zinc, lead and silver orebodies. The Lead Mine and the Copper Mine are distinctly different geologically, but share services including common shaft and surface decline access.

Ground support practices have varied in underground excavations with rock characteristics, excavation size, technology, exposure time and environmental conditions.

2 GEOLOGY

2.1 Lead Mine

The Lead Mine zinc, lead and silver orebodies are wholly contained within the Urquhart Shale, which is a dolomitic carbonaceous siltstone rich in pyrite, and locally in sulphides,

including galena, spalerite, pyrrhotite and chalcopyrite. The deposit comprises of 30 separate mineable stratiform orebodies varying in width between 3 metre and 45 metre from 100 metre to 900 metre in strike length and can exceed 800 metre down dip (Forrestal, 1990). They have an echelon arrangement, striking north south and dipping to the west at generally 65 degrees. The footwall orbodies on the east generally extend further to the south and down dip (Mathias and Clark, 1975). This is illustrated in Figure 1.

The Lead Mine orebodies are divided into the Blackstar (hangingwall) and Racecourse (footwall) orebodies. Typically the Blackstar orebodies are massive and pyritic, with widths ranging from 10–45 metre. The Racecourse orebodies are narrower and lenticular, with widths ranging from 3–15 metre.

2.2 Copper Mine

The Copper Mine orebodies consist of 6 main orebodies, viz, 500, 650, 1100, 1900, 3000 and 3500 orebodies. These are contained within the same stratigraphy as zinc, lead and silver orebodies, however the styles of mineralisation differ greatly. All copper ore is hosted within a silica-dolomite alteration halo that extends concordantly with bedding updip from the Paroo Fault (Perkins, 1990). This basement contact zone varies in thickness and orientation from a tick near horizontal plane to a thin near vertical tight contact.

3 LEAD MINE GROUND SUPPORT

3.1 Historical development and practices

Mining methods in the lead mine have consisted of cut and fill, sub level caving, sub level open stoping, benching and core and shell.

The need for adequate ground support in cut and fill stopes was critical because men and equipment were constantly exposed within the stopes. Cut and fill mining or flatbacking (MICAF—modified incline cut and fill and MECAF mechanized cut and fill) involved maintaining large openings up to 10 metre wide and 13 metre high and over strike lengths up to 1200 metre. Although the rock was generally competent, induced stresses and geological features in the area could lead to instability in some areas. The method also created highly stressed crown and rib pillars.

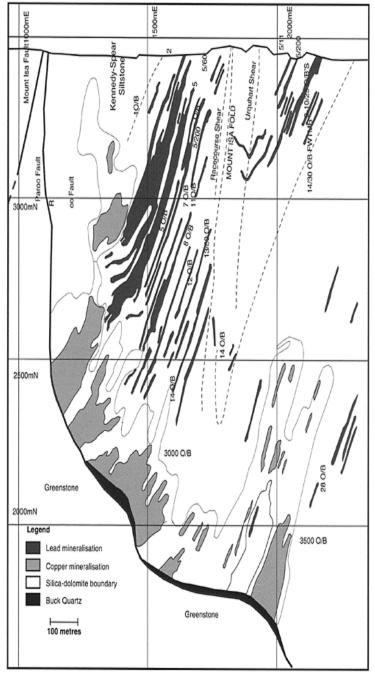


Figure 1. Geological cross-section through 7060 N.



Figure 2. Typical Flatbacking operation.

To minimise the risks a ground support system was developed that involved placing ground support as part of the mining cycle. High tensile point 16 mm anchor bolts (HT bolt), varying in length from 1.5 metre to 3.0 metre were initially used in the footwall, back and hangingwall. Figure 2 shows rockbolt installation.

This bolt provided immediate active support if installed correctly. Bolts were installed by using a hand held rockdrill. The operator would position himself such that he always worked under supported ground. The bolt was used in conjunction with a flat plate, a "Brown" bearing plate, a high tensile washer and 16 mm nut. A compressed air driven impact wrench was used to tension the bolt to 2.5 tonne and permit the "Brown" bearing plate to be compressed to the first flat. The Brown bearing plate acted as an indicator of increasing load after the initial bolt installation. At 3.5 tonne the plate would be flattened to the second flat. Black mesh, 150 mm×150 mm×3 mm gauge, was installed as surface support to the backs to protect operators from scats. In more fissile hangingwalls the HT bolt was installed with a MICAF strap. This steel strap was 5 mm thick, 150 mm wide and 6 metre long with several holes located along the strap to allow bolts to be installed through the strap.

If movement of the hangingwall occurred following firing and prior to bolting, a resin dowel would have been used. Resin grouted 24 mm paddle bolts were installed by hand in this situation. These bolts were also used in development associated with sub level open stoping and installed using a Atlas Copco BUK 11R "glue bolter". This rig was first introduced in the early 1970's in a modified form and was the first step towards mechanizing the ground support process. Fully encapsulated resin bolts requiring the annulus to be filled meant that hole length and cavities needed to be considered. In this situation a plate, washer and nut were tightened on the bolt end. In addition to the resin bolt, a fully cement grouted untensioned 20 mm corrugated dowel had been used since

the early 1970's in access drives, drawpoints and cross cuts in sublevel cave and open stoping methods. Development maybe bolted and at a later date, barred down and meshed prior to stope production.

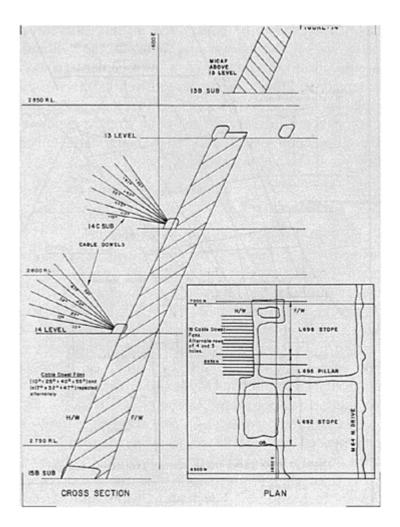
3.2 Cable bolting

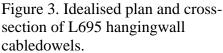
Cable bolts were introduced to Mount Isa in 1973, originally as a support system for crown pillars in cut and fill stopes. At that stage, all bolts were tensioned before grouting. This was time consuming and led to 24 mm cement grouted dowels being installed. The success of cablebolting in recovering crown pillars lead to the application in other areas including support of potential wedge failures, hanginwall support in cut and fill and open stopes and crown and rib pillar support in open stopes (Robertson, 1979).

In 1976, the planning of a new stage of extraction commenced and a decision was taken to use sub level open stoping where cut and fill methods had previously been used. Plans included the use of long cable dowels to reinforce some hangingwalls and narrow crown pillars. Concentrated bands of 15.2 mm diameter, 7 wire cables were installed across the hanging wall, at right angles to bedding, from the stope drill drives. This is shown in Figure 3.

Cable dowels were expected to act by simple suspension but more importantly by binding the bedded rock together and inhibiting bedding plane slip, which was seen as an essential part of the failure mechanism. Previous experience of open stoping similar orebodies had been more conservative or been applied in more favourable ground conditions. Initial results where unsatisfactory, particularly in preventing crown pillar failure. In 1981 it was decided to test modifications to the cable dowelling technique in its application to hangingwalls (Bywater, 1983). Internal research continued and in association with participating sponsors through the Australian Mineral Industries Research Association (AMIRA).

Cable bolts were initially pushed up 57–70 mm diameter holes by hand. Collars of holes where sealed with a tapered wooden plug and/or cotton waste. Cables were grouted with a two-part cement and one part water mix. A 10 mm bleeder tube was pushed to the end of the hole and indicated when the hole was full. The cables were not pre-tensioned, and no fixtures were used along the cables or at the collar. Hand installation was initially replaced with a Gardener Denver carrier with a pushing boom and another boom with a remote controlled basket for carrying two men. The basket assisted in bolt, plug and breather tube installation (Robertson, 1979).





By 1981, 11 stopes and pillars supported with cable dowels, had been mined. The conclusion from this experience was that all failures occurred in shale and not silica dolomite hangingwalls. Bedding planes appeared to have the dominant impact on stability and none of the cable dowels appeared to be effective in preventing or reducing significant hangingwall overbreak. It was concluded that the frequency of bedding planes played a dominant role in influencing the size and number of failures. An improved understanding of the failure mechanisms and cable bolt behavior led to the connecting together of a combination of cables and resin grouted rockbolts pattern with MICAF steel straps. Barrel and wedge anchors were used to fix the straps to the twin cable strands.

These significant findings resulted in the first successful use of cablebolts in open stope hangingwalls (Bywater, 1983).

3.3 Further development

In 1981 a major study of ground support methods was conducted and concluded that mechanical point anchored rockbolts was still the most practical form of back support in cut and fill stopes. Friction anchor split set bolt, 33 mm in diameter was the most suitable form of support for the footwall and hangingwall of

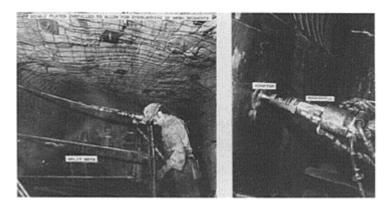


Figure 4. Installation procedure for split sets.

cut and fill stopes but were less suited to areas where there was potential for large wedge of rocks to form in the back. Figure 4 shows the installation procedure and note the addition of mesh not installed in Figure 2. Support characteristics were also improved with the use of a large 400 mm×280 mm steel "butterfly" plate in fissile hanging walls.

Resin bolts were found to be less suitable for some ground support due to; variable properties of the resin after exposure to high temperature, poor quality of installation in fractured ground and the system relied heavily on operator judgment for mixing. The 1981 study also concluded that 20 mm dowels, when grouted with a blow-loaded mixture of sand and cement were less suitable than pumped cement grout because of quality control problems.

The study also concluded that Swellex bolts were to be adopted as the primary means of temporary support. After a testing and evaluation program, fullscale introduction of 2.1 metre, 38 mm Swellex in zinc-silver-lead and copper orebodies was completed in 1985 (Morland, 1985). The exception was in cut and fill stoping where a combination of split sets bolts and point anchor HT bolts continued to be favored for practical reasons. Unlike resin, the effectiveness was not influenced by fractured ground or operator judgment. Swellex bolts installation also leant itself to mechanization and were installed using a Tamrock automatic bolter and the BUK 11R. The Tamrock bolters had also been used to install resin grouted bolts; pumped cement grouted dowels and split sets.

In 1986 when the 15B sublevel to 13 level cut and fill block was approaching existing orebody development on 14 level and recognizing that as the stoping would eventually approach 13 level where stresses would be concentrated, it was considered prudent to trial a different mining method. The advantages of shifting people out of the orebody, larger scale mining and cost reductions appeared to out weigh the disadvantages of the highly selective closely supported cut and fill method. This non-entry method became known as benching. To reap the benefits and overcome the disadvantages, an improved understanding of the ground behavior and stope stability, lead to further changes to ground support practices including adaptation of the knowledge gained in the earlier work described previously in the cablebolting practices of open stope hangingwalls. Birdcaged cable bolts were developed as a stiffer system with more effective and reliable load transfer characteristics (Hutchings, 1990). Continued research into different cable bolt performance, grout mix densities, critical hole diameter and the grout—steel bond strength led to the introduction of the Garford bulb cable (Villaescusa, 1994).

3.5 Current Lead Mine practices

Current practices in the Lead Mine are based on the well established principle of recognizing the required purpose of the excavation; i.e. being either a temporary or permanent opening. Other key principles that had evolved over the years include having personnel only work under bolted and surface supported ground; all support installation is mechanized and managing risk in older areas that are sub-standard.

Permanent long-term accesses are supported with 2.2 metre long fully encapsulated posimix resin bolts on a 1.2 metre spacing. Surface fixtures consisting of 150 mm×150 mm×6 mm thick dome plates and hemispherical ball washers are combined with 100 mm×100 mm×5 mm gauge sheet mesh. This sheet mesh is more rigid and able to be pinned initially in position to the back with short split sets. Installing the resin bolts through the mesh ensures that the mesh is pinned firmly to the back.

Temporary shorter-term accesses are supported with 2.4 metre long 47 mm diameter split sets on a 1.2 metre spacing. Surface fixtures consist of 150 mm \times 150 mm \times 4 mm thick split set dome plates and installed with the same surface sheet mesh support as described for permanent access.

In addition to the above support all turnouts are installed with 6 metre long twin Garford bulb cable bolts on a 1.5-2.0 metre×2.0 metre spacing. Surface fixtures consist of 150 mm×150 mm×6 mm thick plates, hemispherical washers and barrel wedges tensioned at 3 to 5 tonne. These are installed prior to developing the turnout. The same cable bolting system is also applied to reinforce commonly occurring deep wedges. Stope hangingwalls are supported with 4.5 metre single strand Garford bulb cables, plated and tensioned.

4 COPPER MINE GROUND SUPPORT

4.1 Historical development and practices

The use of rockbolts as a major means of ground support rather than timber or steel sets increased from the 1960's. The main advantages over sets were seen to be

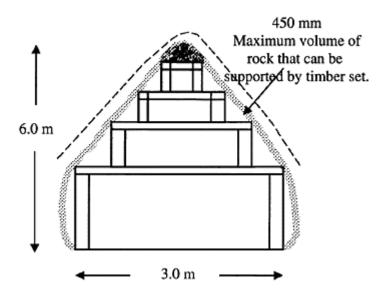


Figure 5. Timber sets for steeply arched drives, indicating the maximum volume of rock that can be supported in steeply arched drive.

lower costs, faster installation, more efficient use of development cross sectional area, and less exposure to blast damage and consequently being able to be installed closer to the advancing face.

The 500 orebody, located in intensely leached silica dolomite, which is generally a very weak material saw this transition. Failure mechanisms in strike drives involved a triangular shaped block with bedding planes as one of the failure planes. Grouted rock bolts 20 mm in diameter, varying in length from 1.8 metre to 4.9 metre in length and arc welded wire mesh was used. Bolt length was determined depending on sidewall corner or back location and drive span width. Rockbolts were often inadequate. Previously used timber sets were also recognized as not being adequate for permanent support in production areas. It was determined that set support should consist of timber legs and steel RSJ beams. They were installed with concrete or timber lagging to allow the load to be distributed on the beam. Sets were installed on a 1.2 metre spacing and RSJ size was increased as spans increased. During mining of development in the 500 orebody it was

recognized that ground support was an integral part of the development cycle (Just, 1966). Figure 5 shows a typical design.

The principle of supporting ground immediately after it was mined was considered sound but very conservative in competent ground in the 1100 orebody. However, emphasis needed to be taken on ensuring washing and barring down was carried out for 50 metre from the face. Supervisors were required to ensure this task was performed and considered to be the most important act of safety. Any competent ground 15 metre outside the hangingwall or footwall of the ore was not bolted except where geological structure, increased stress, or permanent infrastructure determined the need.



Figure 6. Pattern bolting installed well after excavation mined.

The 20 mm corrugated dowel was introduced in the leached ground in the 500 orebody. The term dowel was used where rockbolts were inserted in grout for the full length and were untensioned. Patterns consisted of 3.7 metre, 3.0 metre, 2.4 metre and 1.8 metre dowels in spans up to 6 metre. With the development of the 1100 orebody this dowelling method was used with the length not being altered for some time. Rationalisation of bolt length led to the use of the 2.4 metre pattern in the less competent contact zones and stope extraction horizons. These bolts where also confined to ground that had been previously mined some considerable time before and thus the hazard to the rockbolter had been considerably reduced. See Figure 6.

It was imperative that the correct method of installation was carried out. The cement needed to be of a consistency that allowed full protection against corrosion along the entire length of the dowel. The mixture of two parts screened sand and one part cement was blown around the dowel using a grout gun with water injection and an ANFO pressure-loading vessel. The grout gun was pushed as far up the holes as required to get grout in the back of the hole before grouting commenced. Sufficient thread was left exposed so that washers, nuts, flat plates and mesh could be installed later if required. If plates were installed, only sufficient tensioning was done to allow the plate to just contact the rock.

The 16 mm high tensile was in use for temporary support in the 1100 orebody since extraction was started. It was recognized as being ineffective due to high levels of corrosion in 3 months to 5 years depending on ground water and ventilation conditions.

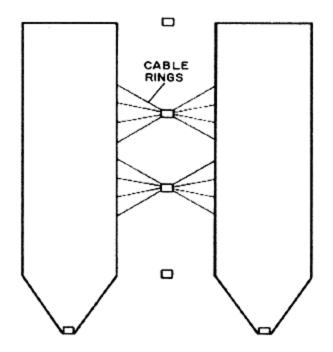


Figure 7. Typical cable dowelling design for transverse pillar support.

It was used to allow development to proceed quickly or in situations where a maximum life of 2 years is required—e.g. drill headings in stopes. It was always used with a Brown bearing plate. A pattern of 2.3 metre long bolts was installed on a 1.2 metre spacing for any span greater than 4.6 metre. A pattern of 1.5 metre long bolts was installed for any span less than 4.6 metres.

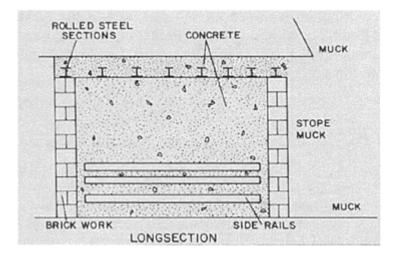
The 24 mm deformed bar grouted tensioned bolt was introduced below 17 level for permanent support in 1970. It was considered to be a high quality bolt used in conjunction with resin capsules for installation in basement contact zones. If resin was

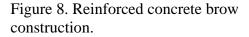
not used then the ground was to be supported using dowels and meshed as soon as practicable after mining.

Weld arc mesh was used in conjunction with the 24 mm deformed bar but was often considered unnecessary or something that could be installed at a later time. Its effectiveness was reduced due to it being placed over plates and not flush against the rock. This was further discussed by Torlach's report on Ground Support in the 1100 Orebody (Torlach, 1976).

Cable dowels were used in the 1100 orebody at this time as a precautionary support in transverse pillars that were required to be permanent access across the orebody. Figure 7 illustrates the principle.

Reinforced concrete brows were installed in most drawpoints throughout the 1980's. Drawpoint conditions were generally poor as the extraction horizons of these large open stopes were mined within the basement contact zone, which is much weaker and geologically complex. Brows consisted of brickwork, preformed steelwork and concrete blown into plywood formed sections (Myers, 1982). Figure 8 shows a typical brow construction.





Shotcrete was restricted to poor unravelling ground and applied before fretting commenced. It was considered to be expensive and labour intensive at the time.

In 1986 a ground support review was conducted at Mount Isa. It concluded that cartridge resins, cartridge cements and blow loaded cement systems were not suitable for mechanized rockbolting. Point anchor bolts like the cone and shell 16 mm HT bolt and splitsets had some limitations to their use but their relative costs made them desirable systems (Grice, 1986). It also concluded that pumped cement dowels and Swellex bolts could be used mine wide in copper and zinc, silver and lead operations. These support systems leant themselves to automatic rockbolting jumbos and would supplement the

existing semi-mechanized Atlas Copco BUK 11R fleet. Concurrent installation of mesh was not available at this time.

Continuing work in the mid 1980's highlighted some of the deficiencies with respect to quality control of cartridge based systems. This included undetectable loss of grout in fractured ground and over drilling leading to incomplete encapsulation, and over spinning or under spinning leading to low strength grout (Grice, 1986). Resin cartridges were also found to have limited shelf life and in some cases were inactive and affected by heat. The blow loaded cement system was shown to contain several defects including low bond strength, poor quality grout and incomplete grouting. Slow gaining of strength also precluded this system from early meshing and mechanization.

As mentioned in the section on the zinc, lead, silver orebodies, Swellex bolts trials and their introduction as the primary means of temporary support occurred in the early 1980's. Limitations to the system were their low shear resistance, high rate of corrosion and cracking of the rock around the hole collar. Following a major rockfall on P45 south drive in 1986 and an investigation to the failure mechanism, cement grouted dowels were recommended to be installed within 2 weeks of Swellex installation. Brown bearing plates were to be installed to permit inspection for increasing loads. Further testing resulted in changing rockboting equipment used to install Swellex bolts to allow mechanized installation of high-tension bolts and split set bolts.

The practice of installing high tensile rockbolts as temporary support in the back of excavations and a grouted rebar and cable bolt system for permanent support continued. In the early 1990's the hollow groutable rockbolt was tested in Mount Isa (Villaescusa, 1992). This bolt gained good acceptance initially with instant support available and a campaign of grouting to be conducted at a later date to provide a permanent system. This appeared to overcome the often doubling up of support systems. However it was found that small movements of ground and failure to systematically grout made the process difficult and impracticable and the installation of this bolt was stopped.

In 1998 and 1999 a fresh assessment was made of resin bolting systems. Trials were completed with the Posimix bolt, Solid Deformed bolt (CS4), the Dywidag bolt (TB2) and the Paddle bolt (Slade, 2002). The fully encapsulated resin bolt provided a number of benefits including immediate high strength active ground support to advancing development and full encapsulation provided significantly greater resistance to corrosion. On the basis that operational quality controls checks are put in place it was able to provide a much safer environment for operators to work in when used in conjunction with mesh. Development jumbos could now be positioned under the last row of bolts and mesh completed from the previous round and progressively install a new row of resin bolts through a new sheet of mesh and the operator would always be working under supported ground a safety principle well learnt during MICAF stoping many years earlier. The Posimix bolt was identified as the most suitable to the operations and provided a one pass permanent system that overcame the issue of slow heading turnaround.

Trials continued in 1999 with the point anchor pumped cement encapsulated Dwydag system.

4.2 Current Copper Mine practices

Current practices in the Copper Mine are based on the same principles as the Lead Mine i.e. being either a temporary or permanent opening and having personnel only work under bolted and surface supported ground. All support installation is mechanized.

Permanent long-term accesses are supported with 2.2 metre long fully cement grout encapsulated PAG (point anchor rebar with a specially designed shell) bolts installed on a 1.2 metre spacing. Surface fixtures consisting of 150 mm×150 mm×6 mm thick dome plates and hemispherical ball washers are combined with galvanised 100 mm×100 mm×5 mm gauge sheet mesh. Fully encapsulated resin posimix bolts, 2.2 metre long, installed on the same spacing are currently being trialled.

Temporary shorter-term accesses are supported with 2.4 metre long 47 mm diameter split sets on a 1.2 metre×1.2 metre spacing. Surface fixtures consist of 150 mm×150 mm×4 mm thick split set dome plates and installed with the same surface sheet mesh support as described for permanent access.

In addition to the above support all turnouts are installed with 6 metre long twin Garford bulb cable bolts on a 1.5-2.0 metre×2.0 metre spacing. Surface fixtures consist of 150 mm×150 mm×6 mm thick plates, hemispherical washers and barrel wedges tensioned at 3 to 5 tonne. These are installed prior to developing the turnout. The same cable bolting system is also applied to reinforce all deep wedges and drawpoints and areas in basement contact zones with additional measures such as shotcrete.

In higher corrosion areas galvanized products are used. In areas of poor ground, areas requiring rehabilitation and drawpoints, shotcrete is being applied.

5 FUTURE DEVELOPMENTS

As the Lead Mine and Copper Mine operations strive to continue to improve safety and productivity new approaches to ground support will continue to be evaluated. The Copper Mine in particular is getting deeper and stress redistribution is becoming more significant. Decline access to the operations has logistically made the use of shotcrete easier. The use of shotcrete to rehabilitate areas with high backs is increasing significantly (Grant, 2000). Shotcrete combined with hydro scaling allows a new approach to continuous ground support in the development cycle and a potential replacement for mesh. This will be trailed in the future.

One pass mechanized bolting has developed, as have the appropriate quality controls to ensure a consistent and reliable support system is installed.

6 CONCLUSIONS

The need to improve safety, productivity and economics has resulted in changes to underground mining methods and equipment. This has led to larger excavations, larger equipment and the development of new technology and ground support systems. Extensive research has needed to be done to understand the ground behavior and the effectiveness of new ground support systems that have resulted in changes in practices, safety culture and technology.

ACKNOWLEDGEMENTS

The author appreciates and acknowledges all those rock mechanics, geologists mining engineers miners, ground support crews and researchers who have contributed to the enormous successes achieved at Mount Isa Mines in developing improved ground support practices. Xstrata is also thanked for the kind permission to publish this paper.

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A case study of ground support improvement at Perseverance Mine

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ABSTRACT: The Perseverance Mine is located near Leinster, 370 km north of Kalgoorlie, Western Australia. Mining consists primarily of sub level cave (SLC). All development and infrastructure is located in the geological hangingwall and current mining is 830 m below surface. Underground mining of the Perseverance ore body has always presented significant challenges due to the difficult ground conditions associated with the weak and altered ultramafic rocks. The range of problems include areas of swelling minerals, intense and extremely weak shear zones, relatively high in-situ stress, and in other areas very brittle rocks. Under these conditions rock mechanics input has been and remains a critical element of the mining strategy The ground support regime required to maintain SLC access has evolved over the years due to these very poor ground conditions. Even with significant ground support and reinforcement systems installed, during SLC cross cut development and ore body extraction significant deformations occur (100 to 500 mm). During the time from development to level closure, two and sometimes three passes of rehabilitation are required to maintain safe access. The development of the ground control systems used in the SLC area at Perseverance Mine are presented and future challenges for the safe and efficient development of these areas are discussed.

1 INTRODUCTION

The Perseverance Mine is located 15 km north of Leinster and 370 km north of Kalgoorlie. The ore body was initially mined for 10 years by the Agnew Mining Company using a variety of underground mining methods with mixed success (due to difficult ground conditions). The mine was mothballed in August 1986 at a time of depressed nickel price. The lease was purchased by WMC Resources Ltd in 1989 and an open cut pit was established above the existing mine workings whilst underground development and rehabilitation continued in preparation for full-scale underground mining.

2 SLC OPERATIONS

The Perseverance open cut was completed at a depth of 190 m in 1995. Most of the nickel in the Perseverance resource is contained in the ultramafic hosted disseminated Perseverance ore body which, apart from remnant mining around old stopes, has been mined underground exclusively by SLC (sub-level cave) methods since 1994. Two distinct strategies have been used (Wood et al. 2000).

- On the upper levels, at 375 m to 520 m depths below surface, the draw from the cave was controlled by mining to a shut-off grade. However, deteriorating ground conditions experienced with depth and more disturbed geological conditions (ore body inflection) forced a re-evaluation of future mining at Perseverance.
- A strategy to drop down below the inflection zone (at approximately 600 m depth) to recommence the cave was implemented. A temporary 80 m pillar was left below the previous mined level and extraction on the lower levels was controlled by tonnage in order to evenly draw the column of ore in the pillar down and limit dilution entry. The ore body continues below the "drop down" levels to at least 1120 m depth.

At January 2004 ore extraction by SLC is on the 9760, 9740, 9715 and 9690 mRL's (760 m to 830 m depth). Ore cross cuts are 5.5 m wide, 4.8 m high and are developed with flat backs in the ore zone. Production levels are 25 m apart (floor to floor). The ore cross cuts are set on 14.5 m centres (leaving a notional 8 m wide pillar). It is possible to sub divide the SLC into three different vertical sections, these area related to the ground conditions observed and the metal recovery achieved (Figure 1). Table 1 provides the production summary for the SLC for the past 9 years.

2.1 Geology conditions

The Perseverance nickel deposit is situated in the Archaean Yilgarn Block of Western Australia. The main disseminated ore body occurs within ultramafic rocks set in the intensively deformed eastern part of the Agnew-Wiluna greenstone belt, which is mainly composed of metamorphosed volcanic and sedimentary rocks. The nickel mineralisation occurs as massive and disseminated sulphides hosted by ultramaficserpentinite lithology's (Barnes et al. 1988).

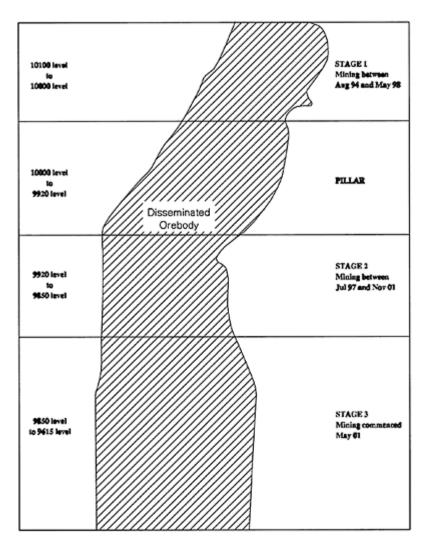


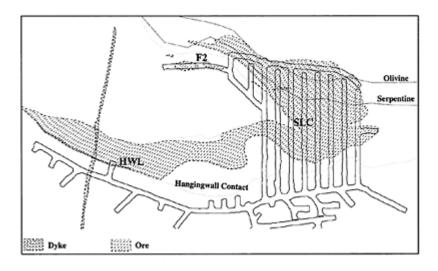
Figure 1. Sub division of SLC into three different vertical sections which are related to the ground conditions observed and the metal recovery achieved (1% Ni contour).

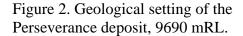
The Perseverance ultramafic is a lens-shaped body several kilometres long and up to 1 km wide in the vicinity of the mine. Within this, the disseminated ore

Mining stage	Actual			Minable reserve			% Reconciliation		
	MT	%Grade	Metal tones	MT	% Grade	Metal tones	Tonnage	Grade	Metal
Ι	1.73	1.41	24394	1.76	1.79	31490	98	76	77
II	5.35	1.77	94686	5.88	1.95	114719	91	91	83
III*	3.14	1.80	56439	3.22	2.16	69665	97	83	81
Total	10.22	1.72	175519	10.87	1.99	215874	94	86	81

Table 1. Perseverance Mine SLC production summary (after Wood et al. 2003).

* Including partial extraction of the 9715 mRL.





body is located on the western margin and is approximately defined by a 1% Ni grade boundary. Typically the ore body is 80 m wide (east-west) and 150 m long (Figure 2). The dip of the disseminated ore body is sub-vertical with an inflection zone between the 10100 and 9900 mRL's (420 m to 620 m below surface) where the dip flattens to around 45° before steepening again below the 9900 mRL (Figure 1).

The hangingwall rocks comprise metasediments and metabasic volcanics, with the dominant rock type a quartzo-feldspathic gneiss. The hangingwall contact with the ultramafic body is marked by a very prominent shear zone containing a mixed assemblage of extremely low shear strength metamorphic minerals (e.g. tochillonite, antigorite). The shear zone is a regional feature that extends to the north of the disseminated ore body to form the hangingwall contact of the Hanging Wall Limb and 1A ore bodies. It thickens in the inflection area of the disseminated ore body.

2.2 Ground conditions

A significant effort has been directed at understanding the distribution of different ground conditions domains, and the key factors that govern the rock mass responses to mining. Ground conditions deteriorated below 10100 mRL (420 m below surface), with the very worst conditions (in terms of floor heave and drive closure rates) observed to date occurring around 10030 and 10000 mRL's (in the centre of the inflection zone at a depth of around 500 m below surface).

Historically the distribution of poor ground in the ultramafic ore body has been related to, or controlled by this inflection zone. Within this broad zone of dilation

	Stress gradient		Dip direction	Stress (MPa) at 9900 mRL	Stress (MPa) at 9400 mRL	
Meta- sediments						
Sigma 1	0+0.0642D*	22	112	39.8	71.9	
Sigma 2	0+0.0367D	28	010	23.3	42.1	
Sigma 3	0+0.025D	52	234	15.5	28.0	
Ultra-mafics						
Sigma 1	0+0.0642D	45	160	39.8	71.9	
Sigma 2	0+0.0367D	45	25	23.3	42.1	
Sigma 3	0+0.025D	24	276	15.5	28.0	

Table 2. Perseverance Mine *in-situ* stress tensor.

* D is depth below surface in m (surface RL at Perseverance is 10520 m).

and shearing, late-stage alteration has affected both the rock mass and discrete structures. Where the ultramafic dip increases again below the inflection zone there is a corresponding gradual improvement in ground conditions, largely due to the hangingwall shear zone, which reduces in width. However, with increases in depth below the inflection zone the ground conditions are affected by increase in the *in-situ* and mining induced stresses and the presence of the barren felsic nose.

2.3 Rock mechanics

The geotechnical conditions encountered at the mine have been discussed in detail in the following references (Wood et al. 2000, Struthers et al. 2000 and Oddie 2002) and are summarised here for the sake of brevity.

2.3.1 In-Situ stress field

A number of *in-situ* stress measurements have been undertaken at Perseverance Mine using HI cells and hydraulic fracturing. These cover the complete range of operating depths down to 9400 mRL (11 Level).

The results for the measurements show a general increase in stress with depth. There is a wide variability in stresses mainly due to the interaction of structural and geological stresses and evidence of this can be observed underground, especially in weaker areas. In general the magnitudes of the stresses conform to an approximate linear increase with depth (Table 2) which is typical of the Yilgarn greenstone belt. The dips and bearings of the measured stresses are consistent within the hangingwall metasediment domain and within the ultramafic domain. The stresses used in modelling are therefore stated in terms of these two domains.

2.3.2 Structure

The rock mass in the disseminated sulphide SLC mining area is routinely covered in fibrecrete within 24 hrs of exposure to maintain the integrity of the

Set ID	Dip	Dip direction
Set 1	74	172
Set 2	72	242
Set 3	62	340

Table 3. Perseverance Mine structure.

excavation. Mapping of joint structures is therefore not undertaken on a regular basis and is biased by exposure orientation. The data in Table 3 are from ore body exposures on 9760 mRL, developed prior to the systematic application of fibrecrete. Joint spacing varies between 0.2–1.0 m and amplitudes of the undulations are between 1.0–3.0 m. Surfaces are generally smooth to slickensided and undulating. Mapping of the hangingwall indicates the presence of three principal defects.

2.3.3 Strength

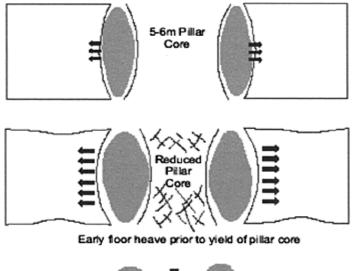
Laboratory tests have been conducted on a number of samples of the major rock types at Perseverance. The tests were undertaken in various accredited laboratories in Australia. The results for the ultramafic rocks and felsics are shown in Table 4. The location of these tests is biased by a relatively large number of tests for the Southern Vent Shaft and for 9920 mRL. The results are indicative of reasonable property values that are expected for these rock types.

3 GROUND CONDITION MODEL

Struthers et al. (2000) discussed ground behaviour in the SLC, particularly the depth of 'rubble zone' in the pillar walls and the rate of associated floor heave. These observations led to a postulated model of crosscut pillar behaviour, as shown in Figure 3. This model

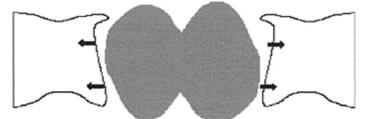
		Density (tonne/m ³)	UCS 50 (MPa)	UTS (MPa)	E (GPa)	v
Ultramafic (Talc-	Mean	2.7	46	4.3	33.7	0.32
Chlorite)	Std Dev	0.14	17	1.3	5.0	0.03
	Count	139	9	5		
Ultramafic (Serpentine-	Mean	2.78	90	7.1	38.8	0.31
Talc)	Std Dev	0.18	23	1.4	7.8	0.07
	Count	109	50	41		
Ultramafic (Olivine-	Mean	3.1	126	12.1	79.7	0.33
Serpentine)	Std Dev	0.4	35	4.3	34.3	0.09
	Count	85	14	11		
Meta-sediments (Felsics)	Mean	2.45	142	13.3	80.5	0.27
	Std Dev	0.16	61	6.1	11.4	0.06
	Count	72	54	59		

Table 4. Perseverance Mine - intact rock properties.





Pillar core reaches critical dimension, and gradually "punches" into floors and backs of adjacent crosscuts



Pillar core yielded, and load shed to adjacent areas. Rate of floor heave markedly reduces. Time dependent deformation continues.

Figure 3. Conceptual ground behaviour model of SLC crosscuts (after Struthers et al. 2000).

is still considered valid. The relative timing between different stages of failure along crosscuts does vary, depending on local ground conditions; the basic sequence of failure is generally the same. It is possible to crudely infer the failure state of crosscut pillars by the rate of floor heave at that point, in the adjacent crosscuts. When the rate of floor heave at any point passed its peak and reduced, it was reasonable to assume the pillar core at that location had yielded, with consequent load transfer to adjacent areas with intact pillar cores. The rate of floor heave in crosscuts adjacent to the latter would then increase, and the cycle repeat. This behaviour has been traced across levels.

3.1 Ground condition monitoring

The principal ground behaviour monitoring technique in the SLC ore body at Perseverance is 2-point convergence measurements. Historically, results from rod extensometers have been erratic, probably due to shear movement across the rods. Monitoring of convergence (closure) across development excavations commenced in the disseminated ore body in late 1996. Closure stations are measured bi-monthly using a tape extensometer with the results graphically reported monthly to mine management. Areas with high closure rates correlate well with identified zone of poor rock mass quality (from drill hole data) and associated weak cross cutting structures (picked up during routine face mapping). Closure to March 2004 is shown in Figures 4a & 4b for the 9690 mRL (830 m below surface).

Over time, the drive width in the SLC crosscuts has varied from 4.5 to 5.1 m, while the pillars have varied from 7.5 to 12.5 m in width. The combination of these two numbers on any single level can be

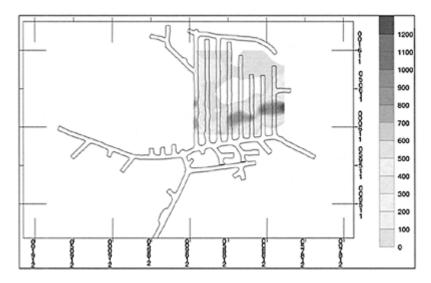
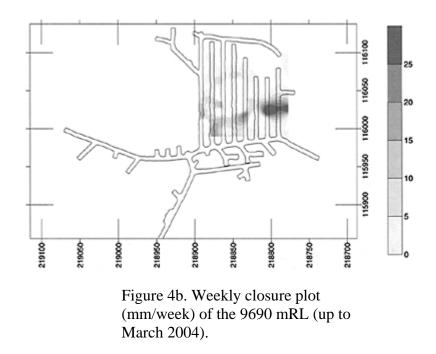


Figure 4a. Cumulative closure plot of the 9690 mRL (up to March 2004).



expressed as the percent extraction of the ore body footprint.

% extraction of footprint=(Drive width/ (Drive width+Pillar width))*100.

This value has been calculated for all the levels and is shown in Figure 5 (for all data to March 2004). The same graph also shows the 'average' drive closure rate for each level. Even though the average closure for the levels is a fairly 'fuzzy' number, it does illustrate the expected correlation between drive and pillar widths and the performance of the crosscuts. In this case a trend of increase in average drive closure can be seen below the 9740 mRL.

4 BRIEF HISTORY OF GROUND SUPPORT SYSTEMS

Prior to 1995 the ground support in the SLC ore crosscuts consisted of Split Sets and mesh installed routinely several cuts behind the developing face. However, on the 10130 and 10115 mRL's (390 and 405 m below surface) there were two 20 to 30 tonne falls of ground within the unsupported section of the drive.

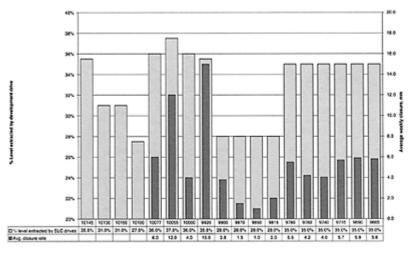


Figure 5. Average weekly closure per level vs % level extracted by SLC ore crosscuts to March 04.

These falls of ground resulted in the decision to install the mesh and split sets right up to the face and support in the development cycle. In early 1995 it was recognised that cable bolts would be required in intersection and spot bolting of badly sheared hangingwall contact areas where required. By mid 1995 the results of initial stoping on the 10130 and 10115 mRL's indicated that the production brows were not sufficiently reinforced to prevent brow break off. By mid 1995 there was a plan to routinely cable bolt the backs of drive, and within 6 months the worsening ground conditions below the 10100 mRL (405 m below surface) made it apparent that the walls would require cabling as well. The cable bolt rings were initially installed on a 2.5 m spacing, this was (within a year) reduced to a 1.25 m spacing.

Post 1995, numerous changes and modifications to the ground support/reinforcement systems used in the SLC ore cross cuts have occurred, these are shown in Table 5.

5 FUTURE CHALLENGES

As the SLC is extracted to greater depths the ground conditions are expected to worsen. The current ground support and reinforcement system used in the SLC has been developed over seven years (see Table 5). It is recognised that any further improvements in the ground control systems used in the SLC will not prevent drive closure and floor heave, but may maintain (the status quo) or even reduce the rehabilitation requirements. The 9715 mRL (805 m below surface) has required approximately two weeks rehabilitation works per ore crosscut. For calendar year 2003, the direct cost of rehabilitation works in the SLC was 20% of the total ground support costs of the mine.

The challenge for the operation is to meet budgeted nickel tonne production while mining a diminishing resource in an aggressive geotechnical and geological environment.

5.1 Details and rational behind current support system

There are three different ground support/reinforcement systems and two drive profiles currently used in the SLC crosscuts (as at January 2004). These are identified in Figures 6a, 6b and 6c. Figure 6d shows the location of the support.

The systems have been designed to allow for a high degree of system ductility in the hangingwall contact zone (Figures 4a & 4b). The reinforcement/support system has been designed around the available site equipment and the knowledge/experience and skills of our

Date	Ground reinforcement	Surface support	Sequence	Comment
Pre 1995	2.4 m friction bolts	F51 weld mesh	Mesh & bolts installed several cuts behind face	Several large rock falls occurred
Early 1995	2.4 m friction bolts & birdcage cable bolts (back only). 2.5 m ring spacing	F51 weld mesh	In cycle temp bolting with cable bolts installed in campaign	
Mid 1995	2.4 m friction bolts,birdcage cable bolts.1.25 m ring spacing	F51 weld mesh	In cycle temp bolting with cable bolts installed in campaign	Review by SRK indicted rquirement for shotcrete. Used in re-hab areas
Mid 1996	2.4 m friction bolts, plane cable bolts. 1.25 m ring spacing	F51 weld mesh, oversprayed with fibrecrete	In cycle temp bolting with cable bolts installed in campaign & post grouting of friction bolts	Non meshed areas deformed intensely. Changed cable bolt system to plane strand from birdcage
Mid 1997	2.4 m friction bolts, back & sidewall de- bonded gewi bolts on a 1.25 m ring spacing	F51 weld mesh & fibrecrete (125 mm thick composit sandwitch)	75 mm fibrecrete layer, temp bolting & mesh, 50 mm fibrecrete layer. De- bonded gewi installed in campaign	First attempts to generate yielding reinforcement system, improved with time
1997– 2002	Several trials at c	dropping the 2nd lay	ver of fibrecrete	Trails all failed due to relatively poor fibrecrete performance

Table 5. Ground support/reinforcement trials at Perseverance Mine.

Dec 2002	2.4 m friction bolts, back & sidewall de- bonded gewi bolts on a 1.25 m ring spacing	F53 weld mesh & fibrecrete (125 mm thick composite sandwitch)	75 mm fibrecrete layer, De-bonded gewi bolting & mesh, 50 mm fibrecrete layer	First attempts at in- cycle bolting in hangingwall shear
July 2003	Modify drive profile in	n hangingwall shear	zone to an arch back	
August 2003	2.4 m friction bolts, back & sidewall de- bonded 3 m gewi bolts on a 1.25 m ring spacing	Fibrecrete & F53 weld mesh (75 mm fibrecrete post mesh)		Introduction of stage I high toughness fibrecrete

underground operators. It is thought that locally stiffening up the system (e.g. by the use of reinforced shotcrete arches) may lead to load attraction and premature failure in these areas.

5.2 Key shortcoming of systems

In low competency areas that require maximum reinforcement and support, the current ground support systems have the following limitations:

- De-bonding of 3 m grouted rock bolts has, in many cases, allowed the entire sidewall to deform and move into the crosscut.
- Issues associated with improving the heading cycle time due to the inability to install fully bonded rock bolts as part of the development cycle in the SLC cross cuts.
- Sections of mesh reinforcing tear preferentially along the cracks when mesh reinforced fibrecrete comes under load. Without reinforcing, the underlying fibrecrete disaggregates and the rock mass starts to unravel.
- In the current active SLC mining areas three different fibre types have been used in fibrecrete production. Currently, LNO are using Barchip Xtreme 60 mm fibres dosed at 12 kg/m³ (Table 6, Tyler et al. 2003). Note that this was a cost neutral switch.

5.3 Future direction of our ground support effort in high-risk areas

Analysis of the past experiences and the practical limitations within the operational (both physical and cost)

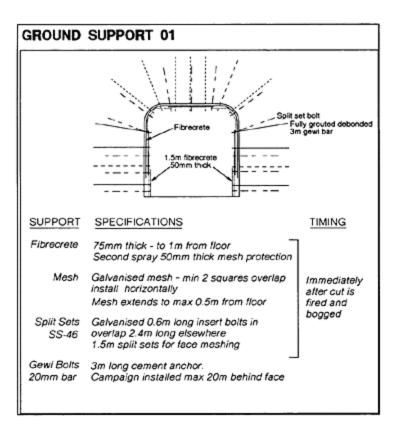


Figure 6a. Ground support profiles installed on the 9690 mRL at Perseverance Mine.

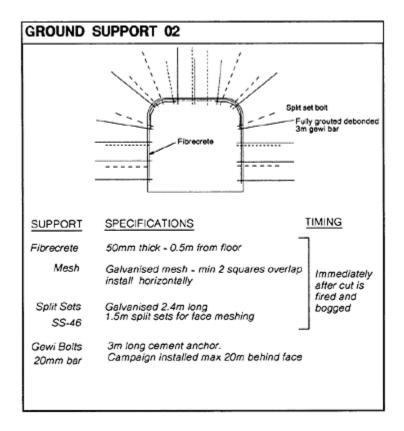


Figure 6b. Ground support profiles installed on the 9690 mRL at Perseverance Mine.

environments suggests the following ground reinforcement philosophy design requirements:

• In February 2004 a limited trial of in cycle resin bolts in the back half of the SLC (in the Olivine altered areas where minimal drive closure is recorded—see Figures 4a & b) was carried out. This trial was highly successful with all bolts installed with no issues and full load transfer developed in the bolts. Additional scaled trials of resin bolts will be carried out in the back half of the SLC. If these prove successful this will effectively allow for a significant improvement in the SLC development cycle time with the associated cost savings.

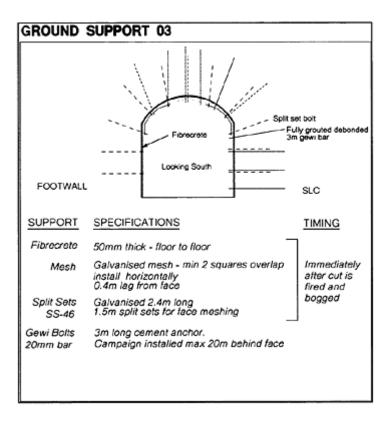


Figure 6c. Ground support profiles installed on the 9690 mRL at Perseverance Mine.

- Continuing improvements in the toughness (ductility) of the fibrecrete. This will enable the use of a more deformation effective lining to act between the surface restraint elements.
 - Further site and laboratory trials are scheduled for calendar year 2004. The mine aims by year end (2004) to have developed and implemented a fibrecrete lining with the toughness parame ters shown in Table 7 (Tyler 2003).
- Review the potential to increase the capacity of mesh reinforcement. This will involve:
 - The use of high strength chain link mesh to replace F53 mine mesh in the SLC crosscuts. Trials of this were initiated in February 2004. Figure 7 shows a typical installation of chain link mesh in the SLC ore body.
 - The use of 7 mm mesh straps installed between the reinforcement elements. Trials of this were initiated in March 2004.

• Increase the depth of anchorage by using fully grouted cable bolts; these higher capacity anchorages will need to be intimately connected to enhanced surface restraint elements, possibly with the use of mesh

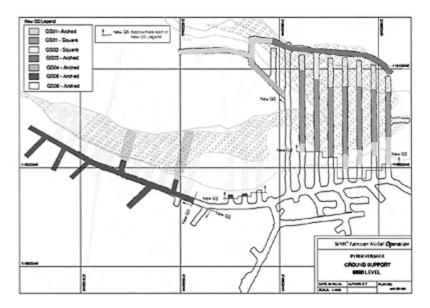


Figure 6d. Ground support profiles installed on the 9690 mRL at Perseverance Mine.

Table 6. Toughness values for different fibres, Round Panel (ASTM C-1550).

	BOSFA Dramix RC65/35	BOSFA Synmix 55 mm	Barchip Xtreme 60 mm
RDP @ 0 to 40 mm deflection	522J	688J	750J
RDP @ 0 to 100 mm deflection	623J	1052J	1140J
% toughness 40 to 100 mm deflection	16%	34%	34%

	June 2004 target	Dec 2004 target	Current site values [#]
RDP @ 0 to 40 mm deflection	>600J (1500)	>750J (1875)	740J (1850J)
RDP @ 0 to 75 mm deflection	>1000J	>1200J	900J
RDP @ 0 to 100 mm deflection	>1200J	>1500J	1100J
% toughness 40 mm to 100 mm deflection	50%	50%	33%

Table 7. Fibrecrete toughness goals.

[#]Barchip Xtreme 60 mm fibre @ 12 kg/m³ (current QAQC test results). Note: Bracketed values are estimated equivalent EFNARC toughness values.

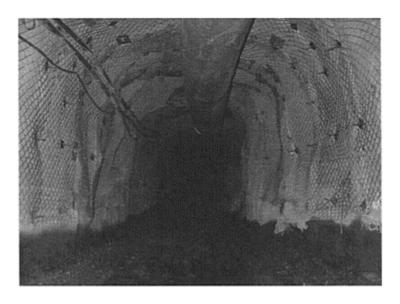


Figure 7. Chain link mesh installed on the 9665 mRL—SLC crosscut 29.

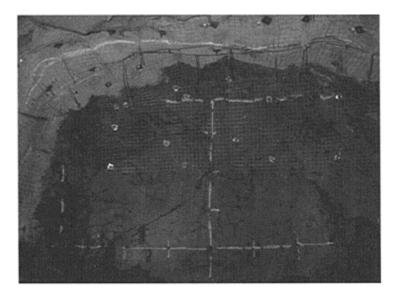


Figure 8. In cycle cable bolting, 8 m wide Progress ore body drive, 9690 mRL.

straps (see above). Trials of this were initiated in February 2004. Figure 8 details photos of the upper drive in the Progress ore body, where 6 m long twin strand cable bolts are installed in-cycle on 1.25 m

- Review of rehabilitation methods: ring spacings.
 - In January 2004 a D6 bulldozer was trialled underground, the D6 was used to rip the floor (in heave affected areas) and strip sidewalls. These initial trials were successful and additional tests
- Review the ore crosscut and associated pillar are being carried out. geometries with regard to metal recovery and associated drive/pillar stability.
 - Note that foundation theory indicates the potential risks associated with damage to the floors and backs of crosscuts if the pillars become too stiff.

6 CONCLUSIONS

- Ground conditions in the SLC ore crosscuts are expected to become increasingly challenging as the ore body is extracted to depth.
- Ground reinforcement/support improvements are continually being reviewed and updated at the Perseverance Mine. Even so, it is recognised that it is not possible (nor sensible) to prevent drive closure and floor heave.

- That the mine has remained operational (and has increased throughput and metal recovery) is a testament to the skill of all those involved, especially the operations staff who deal with severe mining challenges on a day-to-day basis.

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A fall of ground case study—an improved understanding of the behaviour of a major fault and its interaction with ground support

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ABSTRACT: As a result of mining induced changes in ground conditions associated with the extraction of the first stope from the Hangingwall Lens, Copper Mine-South (formerly known as X41 Copper Mine), there was a need to rehabilitate part of the footwall drive adjacent to the stope void. While in the process of scaling loose material from one of the sidewalls within an area of a major fault, a fall of ground occurred which initially buried the booms of a Tamrock Jumbo. Although the failure represented a significant incident, it presented an opportunity to learn about the interactions of the ground conditions (particularly the fault zone) and the ground control systems. This paper will discuss the sequence of events leading up to the failure; the philosophy behind the selection of the original ground support and reinforcement; the philosophy and methodology behind the rehabilitation steps that were adopted once the overall failure had arrested; and the changes made to the ground control practices so as to prevent a similar failure from happening again.

1 INTRODUCTION

1.1 Copper Mine-South

While in the process of rehabilitating the main Hangingwall Lens (HWL) access on 20B Sublevel, a fall of ground occurred. The failure took place in three stages over a period of approximately 8 hours, initiating ahead of the Jumbo and covering the booms, then progressively failing back over the entire unit. Personnel access was restricted following the initial failure, significantly reducing the risks.

With regards to the specific location of the failure, this was along P3849 SEDR on 20B Sublevel, between Q369 CO and Q36 XC (see Fig. 1). The failure initiated approximately midway between the Q369 CO and Q36 XC and arrested to the north of

the P3849 SEDR and Q369 CO intersection. The zone of failure encompassed the area where the J46 fault intersects the main footwall drive.

Originally developed in July 2000, P3849 SEDR (20B) was inspected in October 2001 as a result of ground deterioration in the sidewalls of the drive during the initial stages of production from stope Q369. Production firings were completed in January 2002, with the area re-inspected in February 2002 and rehabilitation recommendations issued.

1.2 General ground conditions in the Copper Mine-South

The Copper Mine-South orebodies extend for nearly 3 km north to south, up to 500 m east to west and vary in depth from 485 m to 1045 m below surface (Grant & DeKruijff, 2000). With production starting in 1966, total production to date is in excess of 160 million tonnes. The copper orebodies are hosted within an Urquhart Shale sequence (with the copper ore occurring as disseminated and massive chalcopyrite), which consists of a 1100 m thick package of thinly bedded black, pyritic and dolomitic shales that typically strike north-south and dip to the west at 65° (see Fig. 2).

The main source of ore is the 1100 Orebody, which starts in the north and extends approximately 2 km to the south, where the orebody then splits into a Hangingwall and Footwall Lens (see Fig. 3). The mining method utilised is sublevel open stoping (SLOS), which has evolved over the years to the present day design standards. Although stope dimensions are typically 40 m by 40 m in plan and extracted to the full height of the orebody (which extends to a maximum of 560 m up-dip), variations of these dimensions are becoming more common as the complexity of the orebody increases.

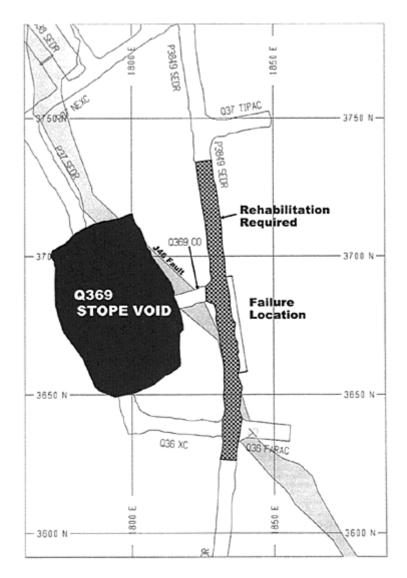


Figure 1. 20B Sublevel mine plan showing P3849 SEDR, surrounding Q369 stope void and development. The grey shaded area represents the floor projection of the J46 fault, and the cross-hatched area represents the area requiring rehabilitation.

1.3 Ground conditions in the Hangingwall Lens

The predominant rock type within the Hangingwall Lens is Fractured Siliceous Shale, with the Basement Contact Zone present in the hangingwall and Silicified Greenstone beyond the contact. Five major faults intersect development at various locations surrounding stope Q369 on 20B Sublevel, namely J46, L41, W41, P41 and the Bernbourough. All the faults can be generalised as having weak rock mass characteristics.

Ground conditions specific to the failure zone consisted of small sized unconsolidated J46 fault material and graphitic shales. The fault material was made up of talc, sepiolite and carbonaceous rubble with buck quartz. The J46 fault strikes in a NW-SE direction, dipping at 58° towards the southwest, intersecting the P3849 SEDR to the south of Q369 CO. The graphitic shales (or bedding) also strike in a NW-SE direction, dipping at 50° towards the west.

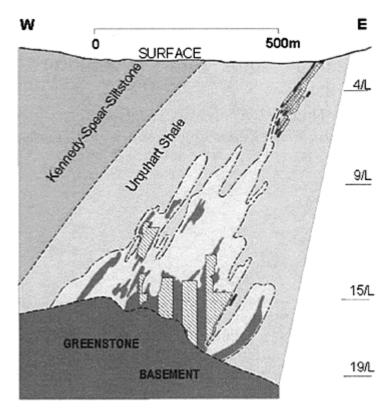


Figure 2. Typical cross section of the Copper Mine-South mining area. The hatched area represents approximately that part of the 1100 Orebody that has been extracted to date.

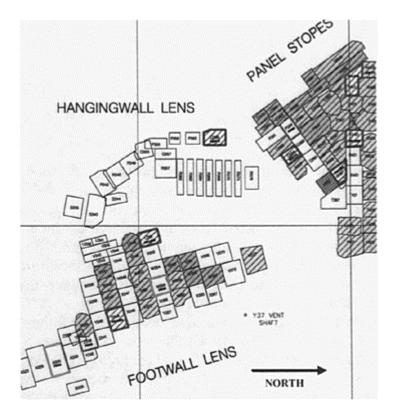


Figure 3. Schematic plan showing the southern end of the Copper Mine-South (the Panel stopes represent the southern most end of the 1100 Orebody).

1.4 Background

Initial work investigating the HWL started back in 1995 (Tyler, 1995) once diamond drilling for the HWL had been completed, after which a series of studies were performed (Li, 1997, Poniewierski, 1998a, b). The main objective of these studies was to determine an economic mining plan. The only significant reference made to development mining was the fact that the geological structural interpretation was only based on drill hole data. As such, there was limited confidence with predicting the ground conditions.

The design strategy for the HWL was to retreat the stoping block south to north, maximising the ore development and minimising secondary pillars. In order to gain some early stoping experience, Q369 stope was targeted to develop a better understanding of HWL behaviour (Q369 was the most northerly stope in the HWL, refer to Figure 3).

Two previous examples were available for review during the design stages of Q369, which had comparable geometries, changes in stress and rock mass characteristics (development on 20B for O383 adjacent to O381 stope and development of S395/S397 stopes on 15 Level). Both cases had a strike exposure of the J46 fault dipping into an open stope. In both cases, the ground conditions had been controlled using rock bolts, mesh and cable bolts. The level of de-stressing was also similar, or greater in the case of O383 development where it was surrounded by fill masses.

As the P3849 SEDR (20B) development mining advanced to the south, new geological information was being gathered and interpreted. As a result, design changes were made in order to minimise the impact and interaction of the major faults on the infrastructure (reducing the number of turnouts along P3849 SEDR and where possible, moving turnout locations away from the J46 fault).

After the development designs were finalised, numerical modelling was performed to examine the potential effects of the HWL mining and the subsequent interactions and effects of faulting on stoping, and the regional effects that the HWL mining might induce as a result of extended relaxation along the faults (Beck, 1999). Observations from the analysis were:

- 1 High potential for fault slip where faults were found in and near stope crowns. The area of slip was sufficiently large, that where faults intersect crowns, some instability should be expected.
- 2 The most significant fault damage would occur on faults that intersect stoping as opposed to faults that are undercut by stoping. The induced fault slip area was greatest on these faults.
- 3 In terms of regional changes, there would be regional softening associated with stoping and the de-stressing of the hangingwall faults may be associated with changes up dip and along the faults towards the X41 Shaft.
- 4 In terms of drive instability associated with destressing from the extraction of stope Q369, the Stress Damage Potential was low (Hudyma & Bruneau, 1998). As such, any ground failures induced through de-stressing were considered to be contained by the recommended support systems installed.

During the design process, and based on historical experience, it was acknowledged that there might be ground deterioration along P3849 SEDR (20B) in the area where the drive was intersected by the J46 fault. However, previous experience of mining through the fault and the subsequently installed ground control systems, had resulted in a stable access being maintained. Such experiences were utilised along P3839 SEDR (20B).

2 WHAT IS A FALL OF GROUND?

2.1 Introduction

At the Mount Isa Mines operations, the definition of a fall of ground is 'an uncontrolled rockfall greater than 1 tonne in size, or an uncontrolled rockfall of any size that causes injury or damage' (Mount Isa Mines, 2003a).

Whenever an excavation is made underground, the surrounding rock mass will react in such a way as to adjust or compensate for the void that has been made (where the reaction tends to relate to rock mass failure of varying degrees). As a result of the ground reactions, falls of ground or rockfalls can occur, where failures can vary in size and consequence (in both cases, this can be from insignificant to catastrophic). As such, falls of ground present a major hazard to the underground mining environment.

2.2 Fall of ground risk management

The risk to underground personnel associated with potential falls of ground is measured in terms of likelihood and consequence (i.e. what is the probability of a fall occurring in a particular instance, and what is the outcome from the fall). In order to evaluate the risk, and then ultimately reduce it to an acceptable level, four steps need to be determined and analysed (with reference to the Standard AS/NZ 4360:1999, 1999):

- 1 Estimate the probability of a rockfall—based on a root cause analysis (the estimation for the probability of a rockfall, whether it is small, large or dynamic, can be based on factors identified from the historical review of falls of ground over a period of time).
- 2 Estimate the exposure to the rockfall—based on the level of activities in a particular area (which can range from high exposure, such as a diesel workshop, to low exposure, such as a barricaded area).
- 3 Estimate the likelihood of a rockfall—the likelihood of a rockfall occurring and injuring a person can be estimated by combining the probability of a rockfall with the exposure. The likelihood is then expressed as either almost certain, likely, moderate, unlikely or rare.
- 4 Estimate the consequence of a rockfall—the consequence of a rockfall, in relation to personnel, can vary from being insignificant (no injuries) to catastrophic (a fatality). Note that determining the consequence of a rockfall will significantly influence the resultant risk rating, and will be driven by the individual (or individuals) undertaking the risk assessment. With conventional risk analysis, the most credible consequence of a rockfall should be used.

Once the likelihood (Step 3) and consequence (Step 4) have been estimated, the risk can be evaluated using a risk analysis matrix. The risk matrix will determine the resultant risk rating (being extreme, high, moderate or low), which in turn will determine the necessary action to be taken to reduce the risk to an acceptable level.

There are many definitions of what a fall of ground or rockfall is. However, there tends to be a common thread to them all, in that 'an uncontrolled failure has taken place'. A suggested common definition of a rockfall was developed as a part of the recent work commissioned by the Minerals Council of Australia and completed by the Australian Centre for Geomechanics (Minerals Council of Australia, 2003a, b)—'An uncontrolled fall (detachment or ejection) of any size that causes (or potentially causes) injury or damage'.

Rockfalls are an ever-present hazard in the underground mining environment, and because of their unpredictable nature, remain one of the greatest hazards to underground personnel. As such, the risk to personnel associated with rockfalls must therefore be managed, which is only possible if a detailed knowledge of the hazard is developed.

In order to assist in developing this knowledge, it is important that all falls of ground are reported. There is a need to determine why a particular failure has occurred, and then prevent a similar failure from happening again. Falls of ground provide opportunities for mine sites to learn more about their ground conditions and ground support and reinforcement, and possibly improve on individual mining practices.

2.3 Falls of ground at Mount Isa Mines

At the Copper Mine (formerly X41 and Enterprise Mines), when a fall of ground occurs, the relevant Supervisor will complete an initial report that provides the Rock Mechanics Engineer with basic details of the incident (only after the area of concern has been made safe to other personnel). The Rock Mechanics Engineer then completes a concise 'Fall of Ground Report' (Mount Isa Mines, 2003b) which considers such information as the failure location, failure size (dimensions and tonnage), induced stress change, failure mode, rock mass quality, excavation details, and ground control details. It is also advantageous, when possible, to photograph the incident and surrounding area to compliment the report.

Falls of Ground in underground mines will continue to occur due to the complex and unpredictable nature of the geological environment in which mining activities take place. Such an environment is further complicated when consideration is given to the extent of material that is commonly extracted over a period of time, an issue which is of particular importance to the Mount Isa Mines underground operations. For this reason, it is the authors' belief that we will never eliminate rockfalls underground.

However, we will eliminate injuries and fatalities due to underground rockfalls as a result of: improved mining practices (for example, mechanised scaling and ground control installation methods); development of mine site procedures and standards in the area of ground control (for example, a Ground Control Management Plan and Ground Control Standards); a continual understanding and improvement in ground conditions and ground control systems; and an improved industry awareness (for example research studies, such as the work being undertaken by the Australian Centre for Geomechanics).

Such a belief can be seen in the industry safety statistics, which show a significant downward trend in rockfall related injuries and fatalities, particularly since 1996–97 (Potvin et al, 2001).

3 EVOLUTION OF GROUND CONTROL PRACTICES AT THE COPPER MINE-SOUTH

3.1 Evolution of ground control practices

Each of the Mount Isa Mines operations requires its own ground support and reinforcement systems, which are tailored to the individual ground conditions and operational requirements. The most effective use of ground support and reinforcement is achieved by matching the ground support to the exposed ground conditions.

Up until 1999, ground support practices involved hand installation methods (cementgrouted rebar, dywidag, and cable bolts, including rolled mesh as a surface support). These systems were a proven and reliable practice with decades of use (Grice, 1986, Potvin et al, 1999). They were simple, robust and low cost systems. However, the practice was inefficient—three pass systems (drill the hole hand held then remove the rig, push the bolts fully encapsulating them with cement grout from a platform, then leave to cure, finally installing a plate).

In addition to the inefficiency, there were several safety issues associated with the systems—working under unsupported ground, working from height (off a platform), manual handling and arduous and repetitive tasks.

Since mid 1999, primary ground support became part of a one pass mining system. The systems adopted were fully mechanised, including the installation of sheet mesh as the surface support. The systems provided immediate support to underground personnel, with reduced residual mining risks and hazards (particularly eliminating the need for exposure to unsupported ground). An additional benefit was an improvement to productivity.

At the time of the P3849 SEDR failure, the primary ground control systems in use at the Copper Mine-South consisted of split sets and mesh for shortterm support requirements, and fully encapsulated cement grouted PAG bolts (or MP Bolts) used for long-term support. The PAG bolt is a point anchored dywidag bolt which provides immediate support via a specially designed expansion shell (Thin et al, 2000). With current practices, primary support has changed in terms of 3.0 metre long cable bolts replacing the PAG bolts for the long-term requirements. Secondary reinforcement remains unchanged and consists of either single or twin strand Garford bulb cable bolts. Like the primary support, secondary reinforcement is also installed mechanically, via Tamrock Cabolters.

Over the years, the use of shotcrete has evolved as a ground control system, gaining increasing acceptance across the Mount Isa Mines operations since the early 1990's. Shotcrete is predominantly used during ground rehabilitation, but has also been used as part of a primary ground control system. Investigations have been carried out looking at shotcrete as a mesh replacement, creating an in-cycle system. However, constraints with providing a constant supply of shotcrete underground (via slick-lines) has so far prohibited this to make it an efficient system (Slade & Kuganathan, 2004). Similar to the current primary support systems in use, shotcrete is applied mechanically, rather than by a hand-held process.

3.2 Ground control installed in the P3849 SEDR (20B) failure zone

The original primary ground support installed along P3849 SEDR consisted of a combination of fully cement grouted PAG bolts (2.2 m long), split sets (2.4 m long) and sheet mesh in the back and down both sidewalls. The intersection of P3849 SEDR and Q369 CO was cabled bolted with 6.0 m long single strand Garford cables.

The total support system installed in P3849 SEDR where the J46 fault was intersected consisted of fully cement grout encapsulated PAG bolts, split sets and sheet mesh in the back and down the entire sidewalls. In addition, 6.0 m long single strand Garford cable bolts were installed in the backs and sidewalls. Such a system has been used many times before throughout the Copper Mine, with ground stability successfully maintained in areas of drives where the J46 fault has been intersected.

Due to the unconsolidated nature of the J46 fault and the graphitic shales, it appeared that the vast majority of the rock had unravelled from around the existing ground support and reinforcement. It was seen that some elements had failed due to corrosion, with J46 fault acting as a path for ground water flow. The level of ground water in this immediate area had been limited to damp ground and not flowing water.

4 SEQUENCE OF EVENTS LEADING UP TO THE FAILURE

4.1 Sequence of events

P3849 SEDR (20B) was being progressively rehabilitated due to ground deterioration in the sidewalls (see Fig. 1). This deterioration was initially observed during the early stages of production from the stope Q369, located 15 metres to the west of the drive (Thin, 2002). An initial inspection of the drive was made by the Rock Mechanics Engineer in order to determine the necessary rehabilitation. As Q369 stope was still an active production source, the drive was barricaded off.

Once the production firings were completed, P3849 SEDR was re-inspected by the Rock Mechanics Engineer. The rehabilitation was recommended to start back just south of the Q37 TIPAC and consisted of scaling loose ground from both sidewalls, then installing split sets and mesh down each sidewall to the floor. The rehabilitation was to continue along the drive moving south. Ground conditions, and subsequent deterioration, was seen to improve past Q36 XC. Cable bolting requirements were to be assessed once the bolting and meshing had been completed (it was anticipated that additional deep reinforcement would be needed in the drive, specifically in the zone of exposed J46 fault). During the last inspection, ground deterioration was not evident in the back of the drive.

Prior to the initial fall, rehabilitation had been completed to the point just south of the P3849 SEDR and Q369 CO intersection. The Jumbo operator during the previous shift had been scaling the eastern sidewall to an approximate depth of 1.5 to 2 m into the sidewall (undercutting the back). The excessive scaling was attributed to the poor ground conditions associated with the J46 fault and graphitic shales. The poor ground conditions were discussed at cross-shift between the day and night shift operators.

The night shift operator continued with the rehabilitation. After nearly two hours into the shift, the operator contacted his Supervisor, concerned with ground conditions on the eastern sidewall adjacent to the Jumbo. The Supervisor inspected the area, which had already been rehabilitated with mesh and split sets. The Supervisor told the operator to pull the Jumbo back and bar down the previously meshed area, after which he was to reinstall mesh and split sets to the floor. The Supervisor then left the area.

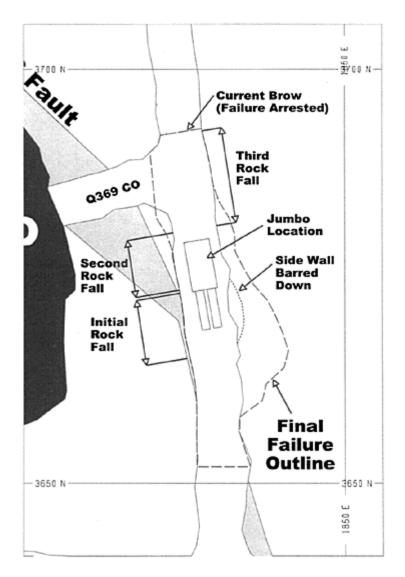


Figure 4. 20B Sublevel mine plan showing details of the sequence of failures and final failure outline. The grey shaded area represents the floor projection of the J46 fault.

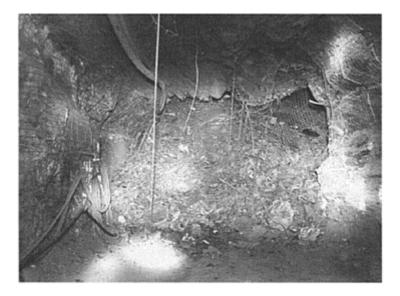


Figure 5. Looking south at the fall of ground at the point of arresting, at the intersection of P3849 SEDR and Q369 CO, 20B Sublevel.

The operator returned to the Jumbo and was in the process of moving the booms into a position to move the unit back. While moving the booms, scats started to 'shower' down. Approximately 2 seconds later the initial rockfall occurred. The operator saw mesh and rocks coming towards him, at which point he took cover behind the console. Once the rocks stopped falling, the operator hit the Stop button and climbed over the right hand side of the steering console and left the unit. The area was then barricaded off.

Various technical personnel inspected the area during the remainder of the night shift to discover the second fall of ground had covered the Jumbo. The third and final fall was discovered just prior to the end of the shift, where the failure was found to have progressed to the P3849 SEDR and Q369 CO intersection (see Figs 4–5).

For a period of approximately 36 hours after the initial failure, localised rock noise was heard in the immediate failure zone (the rock noise consisted of cracking and popping). No further rock noise was heard after this time.

5 PERCEIVED CONTRIBUTING FACTORS WITH THE FALL OF GROUND

5.1 Perceived contributing factors

In order to better assess the immediate failure zone, a 150 mm thick fibrecrete curtain was sprayed on both sidewalls to the floor and in the back (in affect, creating a fibrecrete

arch), to a distance of approximately 6.0 metres back north from the point were the failure arrested (see Fig. 6). Deep reinforcement was then installed in the back and sidewalls of the drive, from the Q37 TIPAC moving south, with the installation of 9 m long Garford cable bolts. Once this was completed, the unstable brow was then mechanically (and thus remotely) removed, after which there was some limited mucking of the failed material. The immediate failure scar and fall material were then visually inspected and assessed.

As a result of this initial rehabilitation and discussions with the relevant Supervisors and operators, several contributing factors were identified which were attributed to the failure. These factors were:

- 1 Under-cutting the back of the drive through deep scaling of the sidewall (initially triggering the failure).
- 2 Continued and progressive mechanical scaling in poor ground.

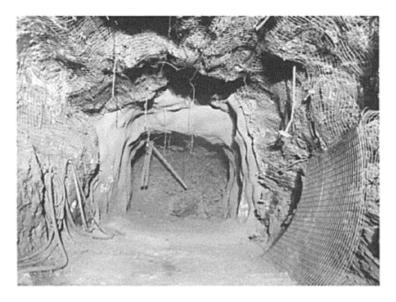


Figure 6. Looking south along P3849 SEDR (20B) at the first stage of the initial rehabilitation process, with the spraying of fibrecrete (150 mm) and prior to the installation of the reinforcing cable bolts.

- 3 A stress window (created by the southern end of the 1100 Orebody and the Footwall Lens), which was subsequently de-stressed due to extraction from Q369 stope.
- 4 J46 fault (and its unconsolidated material properties) and heavily graphitic-coated shales.

The decision as to how to progress with the situation (either full-scale rehabilitation or develop a by-pass) was dependent upon the outcome of the installation of the additional support and reinforcement, and the success of collecting physical facts relevant to the fall.

6 POST FAILURE REHABILITATION PHILOSOPHY AND METHODOLOGY

6.1 Philosophy and methodology

Having safely completed the initial stage of the rehabilitation process, a risk assessment was undertaken in order to determine the next stage of rehabilitation. Access along P3849 SEDR (20B) had to be re-established south of the failure as this represented the production drilling horizon for the HWL.

As part of the risk assessment process, the decision to develop a by-pass around the failure zone was discussed and assessed. While the risks associated with such an action would be lower than those associated with rehabilitating the drive, the question that could not be answered was where did the failure stop in a southerly direction along P3849 SEDR (20B)? The risk of creating an intersection with the by-pass and P3849 SEDR while still in the failure zone proved to be too high, and as such, the option of developing a by-pass was discounted at this time.

With the collection of the physical facts and the level of personnel experience associated with ground rehabilitation, the risk assessment focused on rehabilitating the drive.

Management of weak, friable and unconsolidated failure material drove the basis for the philosophy and methodology behind the rehabilitation (see Fig. 7). The resultant risk assessment identified several hazards associated with a full rehabilitation process of the drive. From this, new controls were identified and a risk management action plan developed. The agreed rehabilitation consisted of:

- 1 Adopting an incremental rehabilitation process, which would limit the amount of exposed and unsupported ground within the failure zone at any one time.
- 2 Adopting a 'project' approach, using operators with extensive ground rehabilitation experience. Two Supervisors were selected and removed from their respective crews. By having these two dedicated personnel, changes in ground conditions would be captured and managed more effectively than if different and rotating personnel were involved. At least one of the Supervisors was present whenever any work was carried out.



Figure 7. Typical size of the failure material from J46 fault exposed in P3849 SEDR, 20B (note that the unit mucking from the failure zone is an Elphinstone R1700).

- 3 Regular inspections by the Rock Mechanics Engineer and updates from the two dedicated Supervisors. Digital pictures were taken of the rehabilitation as it progressed.
- 4 Adopting a rehabilitation cycle, that was flexible and that would be continually assessed during the rehabilitation process (dependent on the exposed ground). The cycle was made up of: remote muck no more than a 4 m advance, then assess; remote spraying of 150 mm thick fibre reinforced shotcrete (mechanical scaling was not used, any loose material was 'scaled' as a result of the impact from the fibrecrete on the exposed ground); install 9 to 12 m long single strand Garford bulb cable bolts (length dependent on ground during drilling) on a 1.5 m bolt spacing and a 1.0 m ring spacing—cables installed remotely with the Tamrock Cabolter; cables left to cure, then manually plated and jacked; then repeat the cycle.
- 5 Communication presentations to the Copper MineSouth workforce. This was an important part of the rehabilitation process as it presented the facts behind the failure, the steps being taken to recover the situation, and the changes that would take place to avoid a similar failure from happening again.

7 GROUND CONTROL BACK ANALYSIS

7.1 Introduction

The design of ground control systems used for development that exposes a major fault has been based on judgement, which has evolved with historical experience over time. This judgement has worked well over the years with drive stability being successfully maintained on many occasions—indeed, there has never been a failure similar to that experienced along P3849 SEDR (20B).

Given consideration to the perceived contributing factors and the experience gained during the rehabilitation, it is believed that the failure was initiated as a result of increasing the effective span in the back of the drive due to the mechanical scaling of the eastern sidewall—exceeding the capacity of the installed ground control systems. Due to the nature of the rock mass in the failure area, the initial failure mechanism would have been that of unravelling in the area of the under-cut back, which then continued and propagated out into the drive.

7.2 P3849 SEDR (20B) ground control back analysis

Empirical design methods exist for assessing drive stability based on rock mass classification, with the Q-system (Barton et al, 1974) being one of the common methods used at Mount Isa Mines. The Q-system is a useful first-pass tool for the design of mine openings. However, it has limitations that need to be appreciated and understood. If any of the Q-system input parameters are incorrectly selected (due to many reasons), the resulting bolting recommendations can be misleading (Misich, 2003).

Due to safety concerns with the exposed ground during rehabilitation (with regards to exposing personnel to the ground conditions), a rock mass classification was not done prior to fibrecreting the fault zone. An estimate was however made based on observations during the rock mechanics inspections and geological mapping carried out during the original development stage (Milne, 2003). As such, the following parameters were used:

- 1 Rock Quality Designation (RQD)=10 (minimum value used).
- 2 Joint Set Number $(J_n)=9$ (three joint sets).
- 3 Joint Roughness Number $(J_r)=2.0$ (smooth, undulating).
- 4 Joint Alteration Number (J_a)=5.0 (alteration between 4.0 and 6.0; 1–2 mm of clay, chlorite, graphite, and clay less than 5 mm respectively).
- 5 Joint Water Reduction Factor $(J_w)=1.0$ (dry, minor inflow).
- 6 Stress Reduction Factor (SRF)=2.5 (single weakness zone containing clay, depth of excavation >50 m).
- 7 Excavation span=5.5 m (original development span) and 7.5 m (final failed span).
- 8 Excavation Support Ratio (ESR)=3–5 (temporary mine opening).

From these parameters, a Q of 0.178 was determined. Then with reference to Figure 8, it can be seen that the empirical estimation of support requirements for a drive with a 5.5 m span equates to bolts and fibre reinforced shotcrete with a thickness of approximately 50 mm. For a drive with a 7.5 m span, this equates to bolts and fibre reinforced shotcrete with an approximate thickness of 85 mm.

Based on this empirical back analysis and engineering judgement, the proposed changes to ground control systems used to maintain drive stability with future exposures of the J46 fault (or any other of the major faults), are to consist of mesh reinforced shotcrete (with an approximate thickness of not less than 100 mm), followed with cable bolts (the length of which will vary from between 6 and 9 m). A cyclic approach of installing this ground control system is certainly preferred over a campaign approach, so as to minimise exposure.

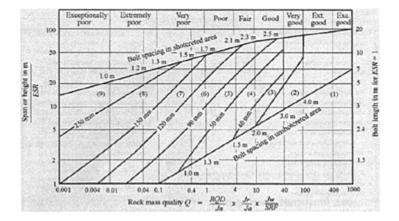


Figure 8. Estimated support categories based on the Q-system (Grimstad & Barton, 1993).

8 ESTABLISHING LONG-TERM HANGINGWALL LENS ACCESS

8.1 Long-term access

Having implemented a practical rehabilitation cycle to allow for the re-establishment of access along P3849 SEDR (20B), the issue of long-term drive stability had to be addressed. Although the effects of future stoping on drive stability were acknowledged, they were not considered as part of the rehabilitation process—both processes were felt to be incompatible with each other due to the complexity of the situation, and had to be dealt with separately.

Extraction of the HWL in a retreating sequence was inevitably going to create stress changes on the failure zone, with a progressively increasing stress path moving towards the zone (the failure zone being located at the end of the retreating sequence). Although the rehabilitation of the failure had created a safe and stable environment, maintaining its stability for the life of the HWL was unknown—would the fibrecrete and cable bolts continue to create a stable environment during the retreating sequence? It was felt that further work would be needed to ensure the long-term stability.

As such, several options were proposed. These consisted of: development of a bypass around the failure zone; installation of an Armco tunnel through the failure zone, then filling the void between the tunnel and failure profile; installation of a shotcrete/concrete

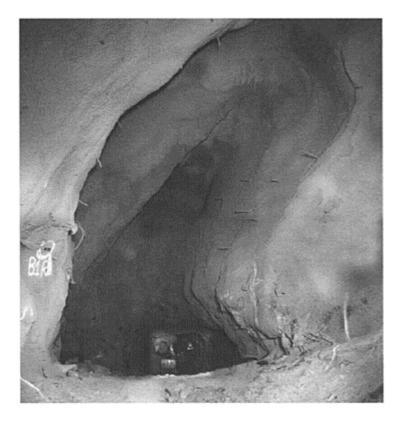


Figure 9. Looking back north along P3849 SEDR (20B) at the failure profile, dominated by the strike of the J46 fault (the photograph has been taken from the top of a 4 m high ramp used during the rehabilitation—notice the vent bag in the top left hand corner of the original drive profile).

arch through the failure zone, then filling the void between the arch and failure profile; and backfilling the failure zone, then mining through the fill.

These options assumed that the level of ground control (fibrecrete and cable bolts) installed during the rehabilitation would be insufficient for the future stress changes. Conversely, consideration was also given to the fact that the rehabilitation would remain

stable. This gave one further option, which was to monitor the stabilised failure zone through instrumentation, and address any deterioration only if it occurred. However, installation of such instrumentation would have limited value, as there was no guarantee that the area would be adequately monitored. In addition, there was no confidence in establishing magnitudes of movement in the installed ground control that if exceeded, would lead to further ground failure.

Having given consideration to the various options in terms of ensuring a safe longterm travel way, while minimising the risk of production delays and cost, the preferred option was the development of a bypass. Although this option was originally discounted during the investigation as part of the initial rehabilitation process, circumstances changed with the re-establishment of the drive—the extent of the failure was seen to have followed the strike of the J46 fault (see Fig. 9). With such information, the bypass could be designed so as to intersect the drive away from the failure. In addition, it was discovered that the bypass could be utilised for future access requirements for the Lower Footwall Lens, information that only became available with the completion of the 2002 Copper Business Study—approximately 9 months after the failure took place (Mount Isa Mines, 2003c).

The bypass (Q37 SEXC and Q36 NEXC) was developed within Fractured Siliceous Shales, intersecting the J46 fault at its southern end (see Fig. 10). The bypass was designed so as to intersect the fault normal to its strike (improved stability when compared to an intersection striking parallel). The development profile through the fault was maintained without any problems, with the installed ground control consisting of split sets, mesh in the back and down both sidewalls to the floor, 100 mm of shotcrete, and cable bolts. The ground control as a whole was installed as a complete system before the next advancing cut was taken.

9 LESSONS LEARNT FROM P3849 SEDR (20B)

9.1 Key learning's

Successful development of fault intersected drives has been achieved many times before, where ground and induced conditions have been similar (if not worst) than those associated with P3849 SEDR (20B). With such cases, stability was maintained with the installation of bolts, mesh and cable bolts in the back and down both sidewalls. Historical experience indicated that such a combination of ground control systems would be appropriate for the ground conditions exposed along P3849 SEDR (20B). However, a significant fall of ground did occur, despite all the correct procedures being followed. So what has been learnt from this failure, so as to prevent a similar failure from happening again?

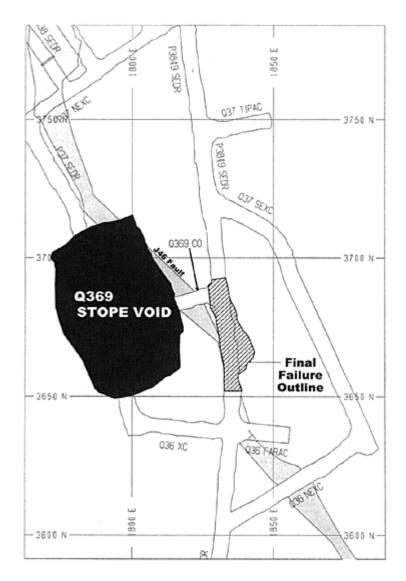


Figure 10. 20B Sublevel mine plan showing the Q37 SEXC and Q36 NEXC Bypass in relation to P3849 SEDR. The grey shaded area represents the floor projection of the J46 fault, and the hatched area represents the failure zone. An extensive and documented design process had been followed for the HWL development and stoping, with a number of group meetings with relevant operational and technical personnel providing input. As a result, several modifications were proposed and implemented prior to development and the start of stope extraction (Grant, 2000).

Poor ground conditions had been recognised in the original development stage, with ground support adjusted to reflect previous experiences of successful development through the J46 fault. Inspections of the drive during stoping resulted in the area being barricaded for personnel safety. The area was then subsequently identified as requiring rehabilitation and a plan developed to undertake this task. The rehabilitation commenced and progressed successfully to the event area.

Change-of-shift communications between operators and supervisors discussed a change in ground conditions seen during the rehabilitation process (this was also documented in the Operator Ground Condition Assessment Sheets). Just prior to the event occurring, the operator had recognised a change in ground conditions and after discussing the situation, decided to adjust the support system being installed.

Given the friable ground conditions where the ground 'fell around the bolts' and the overall weak nature of the ground in the failure zone, the question of effective mechanical scaling needed to be addressed. The power of today's bolting rigs in poor or weak ground could allow such equipment to loosen and remove an infinite amount of material with no real improvement in the ground conditions. Is mechanical scaling in poor or weak ground the most suitable option to take?

With material that has graphitic-coated surfaces, is loose and friable, and likely to move as relatively small blocks, stability should be maintained with tight surface restraint/support—bolt and mesh and/or shotcrete. With loose material of a similar description, should consideration be given to not 'bleeding' mesh that has bagged due to a build up of loose material—more loose material is potentially allowed to move, initiating/propagating a failure? Another layer of appropriate surface support maybe more beneficial.

Such questions, although obvious, are not easily answered and may well be specific to individual sites and situations. In order to help address such questions at the Copper Mine-South (and indeed at all the Mount Isa Operations), changes were implemented to several design and operational procedures:

1 Modifications were made to the Operator Ground Condition Assessment Sheets— OGCAS (Fig. 11, Mount Isa Mines, 2003d). The OGCAS is part of the management of ground control risks, and is a simple process that assists the operator in assessing the ground conditions for their work area, identifying potential ground condition hazards and suggesting necessary action to control them. The sheets are completed for each development or rehabilitation cut advanced. The modifications that were made related to mechanical scaling in poor ground, instructing the operator not to scale more than 1 metre deep, but rather stop and contact their Supervisor. Additional modifications consisted of keeping the sheets in a duplicate book which could be kept on individual rigs, passing on information to the cross shift in terms of what has been installed

LEVEL	NEW DEVELOPMENT REHAB		
LOCATION	What did you install?		
OPERATOR	what do you maan?		
UNIT OROUND CONTROL STANDARD NO.	Bolt Type No. per ring Ring Spacing Sheets of mesh Black or Calvanised		
t. General Ground Conditions	Cracks Present Excessive Loose in Mesh Rocknose		
2. Mechanical Scaling 1	time spert mechanically scaling		
Excessive scaling to	Sidewall Annyingwall Consider Installing more bolls & mesh		
. Is there potential for a wedge to	Profuces large blocks		
	Back Sidewall Hangingwall Footwall		
Ves No			
No 1. Do the ground conditions need	an inspection by Technical Parsannel? vicade heading if needed & inform your Supervisor.		

Figure 11. An example of the Operator Ground Condition Assessment Sheet (OGCAS).

and what issues have been encountered (providing documented history for particular areas).

- 2 Development of the Copper Mine Primary Development and Rehabilitation Checklist— PDD (Mount Isa Mines, 2003e). The PDD is a procedure that outlines the steps involved in checking the engineering details of planned primary development and rehabilitation designs in all the copper orebodies, including the recommendation of the most appropriate ground control for the ground conditions to be exposed. The PDD has input from the relevant Planning Engineer, Geologist, Rock Mechanics Engineer, Ventilation, Development Superintendent and Mine Manager. In addition to the PDD, a separate checklist was developed for areas considered as high-risk rehabilitation (Mount Isa Mines, 2003f).
- 3 Use of risk ratings for existing mine accessways, entry areas and infrastructure as part of the rehabilitation plan for individual areas, covering a total of 275 km of underground development across the lease. The risk ratings were developed through a process of analysis and evaluation of risk, which considered assessing the probability of a rockfall and exposure of personnel to these falls, followed by an evaluation of the consequences of such events.
- 4 As has already been stated, in order to maintain the stability of development that has intersected the J46 fault or other major faults (assuming that the development can not avoid the fault), changes were made to ground control practices in such circumstances. Areas that expose major faults are now supported with a combination of bolts, mesh,

shotcrete and cable bolts in the sidewalls (floor-to-floor), and back—an upgrade on the level of ground control that has historically been installed.

5 Use of numerical modelling to predict fault displacement or changes in mining induced stress in faults zones, associated with stope extraction (Slade, 2003).

Two questions to consider that are pertinent to the use of the upgraded ground control: would the failure have occurred if shotcrete had been applied as part of the ground control system during the original development? And if shotcrete had originally been installed, would it have deteriorated to a point that would have necessitated rehabilitation? Difficult questions to answer. However, with the development of the by-pass comes the opportunity to further improve our understanding of the behaviour of the J46 fault and its interaction with this upgraded support system.

9.2 Proposed ground control instrumentation

As the bypass has intersected the J46 fault, there is the opportunity to install instrumentation internally and externally to the fault. The proposed instrumentation will have two purposes. It should allow for a better understanding of the mechanisms of fault deformation, and also enable drive stability to be monitored with future mining activities (determining how effective the upgraded ground control system actually is).

The proposed instrumentation will consist of (Milne, 2003):

- 1 Borehole Camera Holes, for visual monitoring within the fault.
- 2 Closure stations, installed to determine if ground movement is occurring and if it is being transferred to the shotcrete. The stations can also be used to measure the distance between closure stations (across the drive) to give an indication of shear movement.
- 3 SMART cables, installed to determine if the cables are loading (the SMART cables, coupled with the closure stations, should determine if the cable bolt/shotcrete ground control system is behaving as expected).

10 CONCLUSIONS

Despite the length of time that mining has been in existence, the behaviour of a rock mass in a producing environment still remains unpredictable. This behaviour is then exacerbated when a mine has been an active source for a long period of time (as is the case with the Copper Mine-South).

The failure in P3849 SEDR (20B) was no exception to the unpredictable nature of a rock mass. Such a degree of ground reaction had never been experienced before at the Copper Mine. However, with the work that has been undertaken as part of the rehabilitation process (and the planned future instrumentation program), a significant amount of knowledge has been gained, which will only aid in the overall understanding of the behaviour of our rock mass and its interaction with ground support. Changes have been made to our design process and Development practices, including modifications to ground control systems in areas with exposed faults, that represent a significant move forward to prevent a similar failure from happening again.

For as long as excavations are created underground (both at a development and production scale), falls of ground will continue to occur. The challenge that the industry faces is the effective management of such hazards.

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Field experiments on cable bolting for the prereinforcement of rock masses—first application to an underground powerhouse in Japan

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ABSTRACT: Cable bolting is a support method for the pre-reinforcement of rock masses. It has the potential to strengthen rock masses and to make them more ductile, thus decreasing the number of rock anchors which need to be installed after the excavation. Cable bolting may also bring about an upgrade to the construction efficiency and the stability of underground caverns, such as underground hydropower stations. Nevertheless, cable bolting has not been used much yet in Japan. In this research, field experiments are planned and carried out in order to investigate the applicability of cable bolting to the construction of an underground powerhouse. Numerical simulations are also conducted to better understand the mechanism of the effects of cable bolting from the viewpoint of the path of the rock stress.

1 GENERAL

1.1 Redevelopment of hydropower plants

Hydraulic power generation is renewable energy, with less carbon dioxide (CO_2) gas emission, that can contribute to the stabilization of power sources and flexibly provide electricity during peak hours. Since the demand for electricity widely fluctuates, especially during the daytime, the shave cutting of the peak electricity is an essential issue in Japan. In the future, therefore, it is desirable to develop a steady supply of hydraulic power generation. In addition to the development of continuous hydropower, the effective utilization of existing power facilities, such as dams and reservoirs, is advocated as a strategy which is expected to result in the reduction of environmental loads and construction costs.

The expansion project for the Okutadami Hydropower Station (Electric Power Development Co., Ltd., referred as to the Project hereinafter) is a redevelopment hydropower project that is intended to build up the output of power generation, particularly during the peak hours of the day, by utilizing the existing Okutadami Reservoir. The existing underground powerhouse is extended sidewise to provide a cavern for a new generator unit, namely, Unit No. 4, which is installed right next to the existing underground powerhouse. This concept, including the arrangement of the extension of a powerhouse, is rare; it is the first case of its kind in Japan.

The project commenced in July of 1999 and the power station began operation in June of 2003.

1.2 Cablebolting

The soundness of caverns for use as hydropower plants or oil storage facilities, etc. is secured with the arrangement of pre-stressed anchors, rock bolts, and shotcrete. Post-set supports (dowels, rock anchors, etc.), installed after the excavation, often bear a significantly large load after the completion of an excavation. This is due to the relaxation or the deterioration of the surrounding rock which occurs when a cavern is situated in a weak rock zone. From the viewpoint of the economy and the efficiency of the construction, if weak rock zones are anticipated around a cavern prior to the excavation, adequate preset supports may be able to secure the soundness of the cavern and reduce the number of post-set supports required. It is worthwhile, therefore, to examine the installation of cable bolts for use as preset supports.

Cable bolting is a support method for the prereinforcement of rock masses. It has the potential to strengthen rock masses and make them more ductile.

Cable bolting is similar to the dowel support method which uses steel rebars. The cables are installed in boreholes without tension before/after grouting. A cable bolt is a flexible tendon consisting of a number of steel wires that are wound into a strand. Since cable strands can bend around fairly tight radii, they can provide a high performance, in terms of flexibility and handling, and can enable the installation of long bolts from confined working places. These features facilitate the installation of preset supports in caverns, before the excavation, from small tunnels such as exploratory adits or transport adits located around the cavern. Therefore, cable bolts have been widely used mainly for mining excavations (Stephansson 1983; Kaiser & McCreath 1992; Hutchinson & Diederichs 1996; Broch et al. 1997; Villaescusa et al. 1999).

The design for the cable bolting depends on a conventional method or an empirical method, using rock classifications (Potvin et al. 1989; Hutchinson & Diederichs 1996). A rational design method has not been established because the mechanism of cable bolting has not yet been clarified.

In this research, field experiments are conducted at the construction site of the Project for the purpose of demonstrating the applicability of cable bolts as preset supports for the construction of underground caverns. The rock displacement and the axial force of the cable bolts are monitored during the excavation of the cavern. Numerical simulations are also conducted to better understand the mechanism of the effects of cable bolting from the viewpoint of the path of the rock stress.

2 OVERVIEW OF THE EXPANSION PROJECT OF THE OKUTADAMI HYDROPOWER STATION

2.1 Project description

The site of the Okutadami Power Station Expansion Project, shown in Figures 1 and 2, is situated in EchigoSanzan-Tadami National Park in a mountainous area of Niigata and Fukushima Prefectures in the central part of Japan. The Project generates 200 MW of electric power at a maximum output during peak hours by adding an available discharge of 138 m^3 /s at a maximum and obtaining an effective head of 164.2 m by use of the existing Okutadami Reservoir. The discharge for the Project is managed by the modified operation that concentrates the generation of electricity during the peak hours of the day, while the discharge for the existing power station is not reduced in terms of power, but is reduced in terms of duration.

The existing main civil structures are as follows:

- 1. Okutadami Dam: a straight gravity concrete dam, 157 m high, with a crest length of 480 m and a volume of 1,636,300 m³.
- 2. Okutadami Reservoir: Tadami River of the Agano river system, with a catchment area of 595.1 km², a gross storage capacity of 601,000,000 m³, an effective storage capacity of 458,000,000 m³, and a design flood discharge of 1,500 m³/s.
- 3. Underground Power Station: Unit Nos. 1,2 & 3, with a cavern 22 m in width, 41.30 m in height, 89.6 m in length, a maximum power discharge of 249 m³/s, an effective head of 170 m, and a maximum output of 360,000 kW.

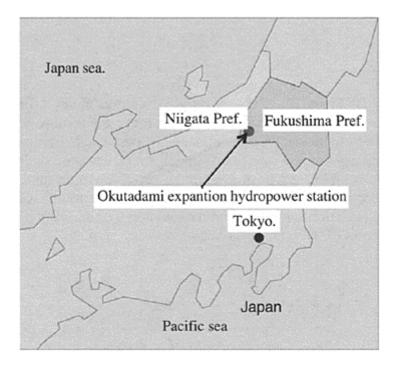


Figure 1. Loacation map of the Okutadami expansion hydropower plant.

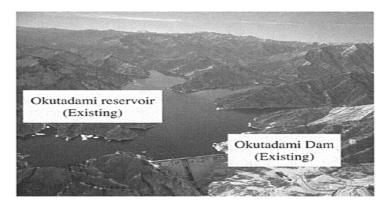


Figure 2. View of the existing Okutadami dam and Okutadami reservoir.

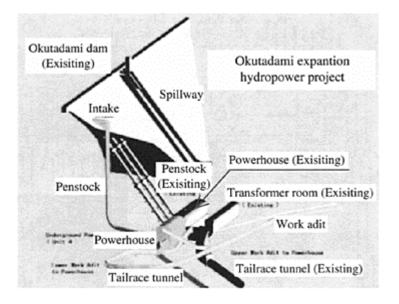


Figure 3. Outline of Okutadami dam and the powerhouse.

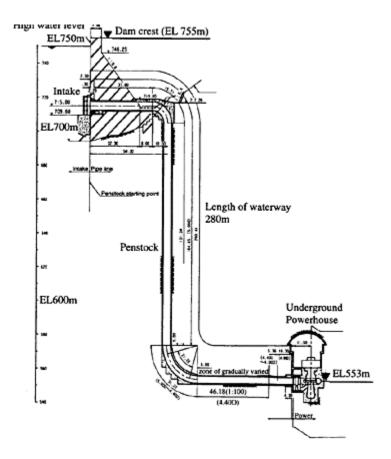


Figure 4. Profile of the waterway.

In terms of the civil structures, the Project features are as follows. The new intake, to be arranged additionally, is constructed collaterally to the existing Okutadami Dam on the right bank of the existing intake, and the penstock conveys water to the underground power station through a vertical shaft and a lower horizontal conduit after passing through the dam body. The additional power station for the new unit, referred as to Unit No. 4, has been constructed by expanding the existing power station sidewise. The main component is the extension of the powerhouse cavern, 46 m in length, 20 m in width, and 41 m in height, which has a mushroom-shaped profile and an excavation volume of approximately 29,000 m³. The non-pressurized tailrace tunnel has been constructed parallel to the existing tailrace tunnel and release water flows from the outlet, located approximately 2500 m downstream from the existing Okutadami Dam to the existing Ohtori Regulating Reservoir.

The outline of the Okutadami Dam and the powerhouse, the profile of the waterway, and the sections of the powerhouse are shown in Figures 3, 4, and 5, respectively.

The technical features of Okutadami Unit No. 4's construction works are as follows:

- 1. The construction period for the Project is about 4 years (1999–2003). Parts of the construction activities are restricted to only 4 months per year (from July to October), not corresponding to the breeding season of the golden eagle which is registered as one of the endangered species in Japan. The actual construction period, therefore, is short.
- 2. The construction works are to be done without stopping the operation of the existing Unit Nos.1, 2, and 3.
- 3. The construction works of the waterway structures, except for the intake and the outlet, are done underground.
- 4. The intake works are done under dry conditions inside the semicircle-shaped cell of the double steel sheet pile and in between thin concrete.

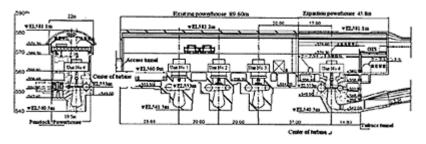


Figure 5. Section of the powerhouse.

- 5. The concrete excavation of the Okutadami Dam, a 6.2 m square with round haunches, is done for the installation of a steel penstock.
- 6. The underground cavern for Unit No. 4 is excavated right next to the existing underground powerhouse during the operational condition of Unit Nos. 1, 2, and 3.
- 7. Existing equipment such as the intake gantry crane, the access tunnel, the overhead traveling crane, and the cable tunnel is also used for the operation of Unit No. 4.
- 8. In the construction activities, various measures for environmental protection are implemented.

2.2 Geology and the support design of the cavern

The host rock in the powerhouse is gabbros and the overburden is about 180 m in depth. Sound rock is distributed over the site of the power plant. The joint system has approximately the same strike along the axis of the cavern. During the investigation phase, primary stress measurements were performed in boreholes. These stress measurements indicated a maximum principal stress of 5 MPa with ratios of two other principal stresses, namely, 0.94 and 0.77, respectively. The original design for the supports of the cavern is as follows:

- 1. 1.0 m thick arch concrete
- 2. Densely spaced fully grouted rock bolts with lengths of 3 m in the roof and 5 m in the walls

- 3. Widely spaced pattern of pre-stressed anchors in the wall for the reinforcement of the abutment portion of the arch concrete
- 4. Fiber reinforced shotcrete in the roof and in the walls.

Figure 6 shows the typical rock support in the powerhouse cavern and the configuration of the excavation of the cavern.

2.3 Cavern bulk excavation using advanced controlled blasting

The main body excavation of the cavern was performed by the bench cut method with benches 3 m in depth. Before starting the operation, the details of the blasting technique were examined to minimize the influence of the blasting vibrations to existing power Unit Nos.1, 2, and 3, which are located at a distance ranging from 7 m to 42 m from the blasting area. The Nonel Blasting System, which is a non-electric detonator system based on a signal line, has been adopted because of its safety, reliability, and controllable function. Using the monitoring results of the first bench blasting (No. 5 bench shown in Figure 6), the vibration data were analyzed to obtain the relation among the vibration velocity, the distance, and the explosive weight per blast hole. Figure 7 shows the results of the correlation analysis, and the correlation formula is shown below. The vibration velocity must have been controlled below the criteria of 2 kine (cm/sec) at the center of Unit No. 3, which is the nearest unit to the construction area.

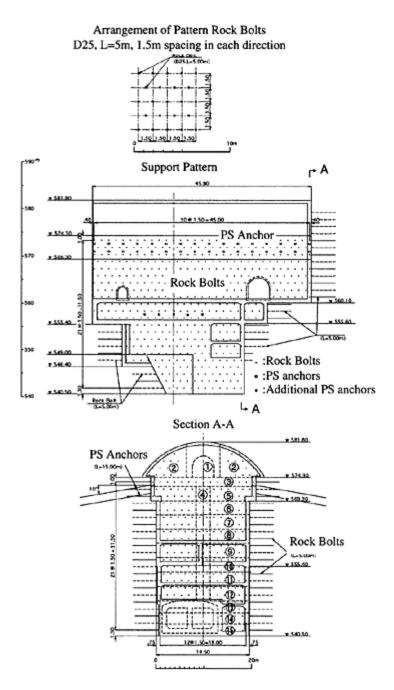


Figure 6. Typical supporting of the powerhouse.

$$\nu = 102.4 \frac{w^{0.75}}{r^{1.538}} \tag{1}$$

where v (kine)=vibration velocity, r (m)=distance, w (kg)=explosive weight per blast hole.

Figure 8 shows a plan view of the blasting area and the vibration monitoring points for the existing powerhouse. The blasting area was mainly divided into 4 blocks and the block next to the existing powerhouse

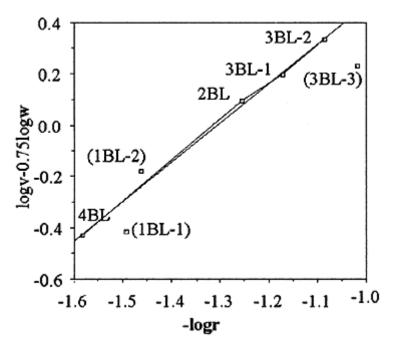


Figure 7. Vibration due to blasting.

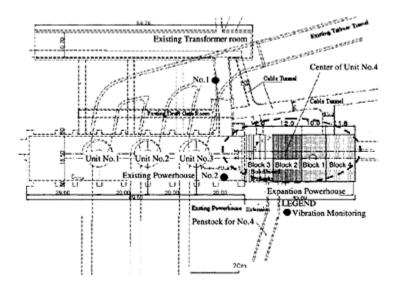


Figure 8. Blasting segment and monitoring for the blasting vibration.

was subdivided into 8 blocks. Therefore, the blasting area in each bench was divided into 11 blocks. A slurry explosive was used for the primer cartridge and Ammonium Nitrate and Fuel Oil (ANFO) were extensively used for the main explosive. All blasting was performed after the daily operation of Unit Nos. 1, 2, and 3. Blasting was carried out for the main body excavation a total of up to 50 times, and the observed maximum vibration velocity was 1.99 kinc.

The maximum explosive weight per blast hole was designed to be below 0.2 kg/delay in the blasting block next to the existing powerhouse.

The excavation of the cavern, 6300 m^3 in arch section and $22,300 \text{ m}^3$ in bench sections, has been successfully completed in seven (7) months with a suspension period of nine (9) months due to the requirements of the preservation of the surrounding environment.

Figure 9 shows the view of the powerhouse cavern at the completion of the excavation.

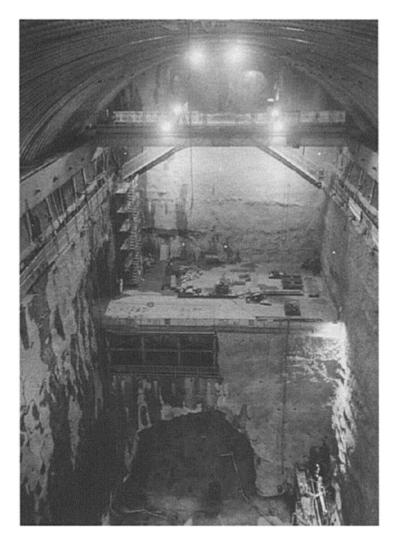


Figure 9. View of the powerhouse cavern at the completion of excavation.

$2.4\ Measurements of the cavern behavior$

Measurements were performed for the crown settlement, the convergence of the walls, the displacement within the rock mass, the axial force of the rock bolts and the arch supports, and the rebar stress.

The largest convergence occurred at the highest section at the center of Unit No. 4. Figure 10 shows the measurement results of the convergence.

The convergence increased markedly after the 6th bench lift was excavated, whereas it hardly increased during the suspension of the construction. Although a slight creep

displacement was seen after the excavation was completed, it converged in several months and the cavern has been stable since then. The point where the largest convergence occurred is located 566.30 m above sea level and the convergence value was 26.4 mm. In the mean time, the excavation of the cavern did not exert any particular impact on the existing power station.

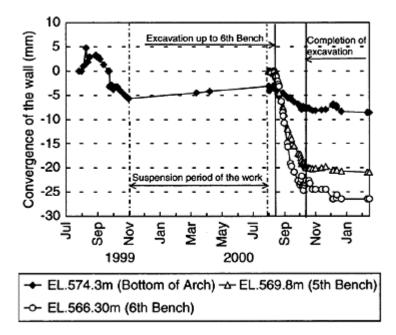


Figure 10. Measurements of the convergence of the cavern during the excavation (the highest section of the cavern).

3 FIELD EXPERIMENTS ON CABLE BOLTS

3.1 Purpose of the experiments

In the experiments on the cable bolts, the following items are studied in order to demonstrate the applicability of the cable bolting method to the stabilization of caverns:

- 1. Evaluation of the effectiveness of the preset supports with cable bolts
- 2. Evaluation of the performance in terms of the type of cable bolts used
- 3. Understanding the mechanism of cable bolting.

3.2 Cavern site and its geology

The experimental region, located in the middle section of the cavern of Unit No. 4, is 10 m wide along a plane between the cavern and the adit (4.2 m wide and 4 m high), as shown in Figure 11.

In terms of the local geology of the experimental region, there are three joints continuously passing through the experiment region. These joints could cause a significant displacement of the cavern wall during the excavation. Therefore, the excavation of the cavern was implemented with the arch excavation and then followed by the bench excavation of twelve benches ranging in height from 2.3 m to 3.2 m.

3.3 Arrangement of the preset cable bolts

The design of the cable bolts involves the following:

1. Type of cable bolts and the number of strands in the borehole

- 2. Length and spacing of the cable bolts
- 3. Kind of grout and its water-cement ratio.

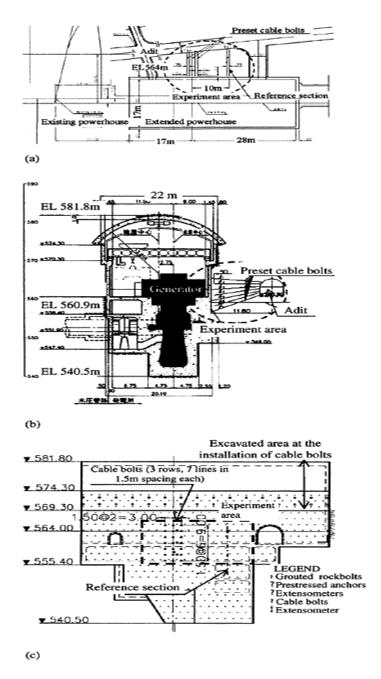
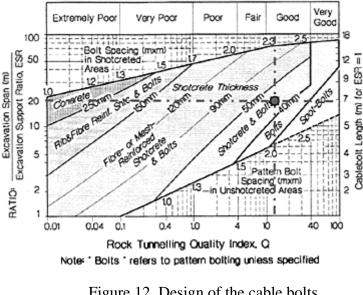
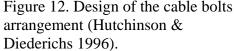


Figure 11. Location of the in-situ experiment. (a) Plan, (b) Transverse section, (c) Longitudinal section.

3.3.1 Type of cable bolts and the number of strands in the borehole

Since cable bolts with two strands in one borehole are the standard application for mines in Canada and Australia, the same arrangement of strands is followed





in this study, as shown in Figure 14. In addition to the two types of 7-wire plain strands, bulbed strands are also applied to compare the effectiveness of each type. Bulbed strands have kinks along them at regular intervals to upgrade the bonding property to the grout. The 7-wire strands are 15.2 mm in diameter and have a yield load of 200 kN per strand.

3.3.2 Length and spacing of the cable bolts

The empirical method of the Q-system (Grimstad & Barton 1993, Hutchinson & Diederichs 1996) is adopted in order to select adequate spacing for the cable bolts. The length of the cable bolts, 10 m, is subjected to the distance between the cavern and the adit.

The Q value at the site is estimated at 12.5 based on the geological conditions at the site, a cavern width of 20 m, and an excavation support ratio of 1.0. This provides a space of 2.5 m for the supports with shotcrete and a space of 1.0 m to 1.5 m for those without shotcrete, as shown in Figure 12. Considering these results, together with the space of the post-set dowels (1.5 m), the space of the cable bolts was also determined to be 1.5 m.

Based on the above considerations, twenty-one sets of cable bolts are arranged from EL 565.5 m to EL 556.5 m at a height of 9 m and in three rows 3 m in width on the cavern wall, as shown in Figures 13 and 16.

3.3.3 Kind of grout and its water-cement ratio

Premixed mortar composed of Portland cement and dry sand is applied as the grout material considering its frequent application at Japanese tunneling sites and its high workability. The optimum grout mixture was studied in laboratory experiments (Ito et al. 2000). As a result, a W:C ratio of 40% to 45% was found to be adequate in terms of the groutability, the strength properties, and the pumpability.

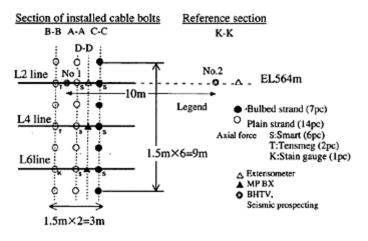


Figure 13. Arrangement of the cable bolts and instruments.

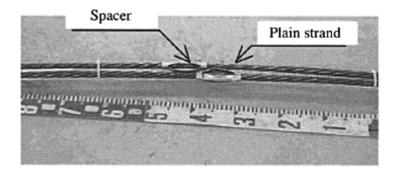


Figure 14. Double strand with spacers of cable bolts.

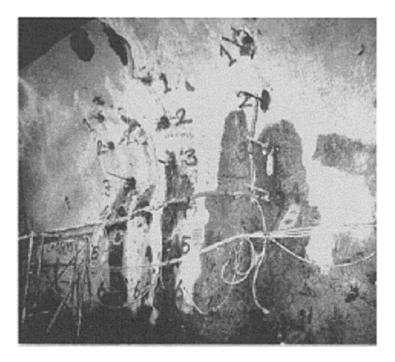
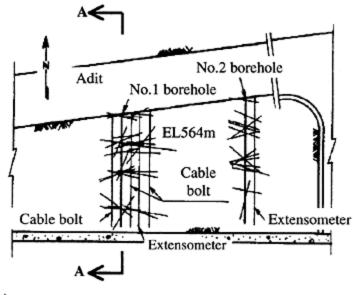


Figure 15. Situation of the installed cable bolts.

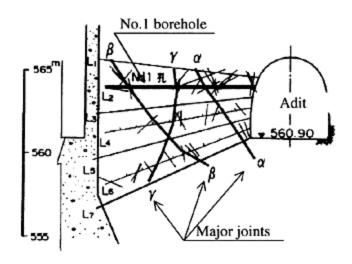
3.4 Monitoring instruments

The purpose of monitoring is to

- 1. Evaluate the effectiveness of the preset cable bolts;
- 2. Evaluate the difference in the effectiveness of the cable bolts, Le., plane and bulbed cables;
- 3. Evaluate the performance of the instruments for the axial force monitoring of the cable bolts.



(a)



(b)

Figure 16. Joint distribution in the experiment area. (a) Plan, (b) A-A section.

For item (1), reference section K, without preset cable bolts, is prepared in order to monitor the displacement of the rock; it corresponds to pre-supported sections A, B, C, and D. To clarify the geological conditions and the joint characteristics of these sections, an inspection with a borehole TV (BHTV) was conducted in each section prior to the cavern excavation.

For item (2), two kinds of strands are adopted, as shown in Figure 13. The displacement of the rock and the axial force of the cable bolts are monitored for each cable bolt.

For item (3), the following instruments are prepared:

- 1. TENSMEG (Rockest Co., Canada, Choquet & Miller 1988): an instrument with the strain wire twisted around the strand.
- 2. Strain gauge type (Toyoko Elmes Co., Japan): strain gauges attached on the thin tube covered with the cable strand.
- 3. SMART (MDT Co., Canada, Hyett et al. 1997): small extensometer installed instead of the king wire of the strand.

Two extensioneters and nine axial force meters (six SMART, two TENSMEG, and one strain gauge type) are arranged in the field experiment region. Figures 14 and 15 show the cable bolts of the plain strands and the situation of the adit after the installation of the cable bolts and the instruments, respectively.

4 RESULTS OF THE MEASUREMENTS

4.1 Joint distribution at the experiment region

Based on the investigation with the BHTV of the boreholes in each region, with and without preset cable bolts, the following features are summarized concerning the joint distribution in the experiment region, as shown in Figure 17:

- 1. Eighteen joints, namely, two joints per meter, are distributed in the borehole in presupported section D, while twelve joints, namely, 1.2 joints per meter, are distributed in the borehole in unsupported section K.
- 2. The joints dominate the strike in an east-west direction and dip to the north at a 50 to 60 degree angle.
- 3. Clear continuous joints a and (3 and unclear continuous joint γ are in pre-supported section D.
- 4. There are no continuous joints in section K. Moreover, there is no obvious continuity of joints between sections D and K.

4.2 Displacements of the rock due to the excavation of the cavern

The distribution of the rock displacements measured by the extensioneter at EL 564 m during the excavation is shown in Figure 18; it compares the pre-supported and the post-supported sections, D and K.

The displacement distributions at both sections remain at almost the same level until the excavation progress at EL 560.1 m, corresponding to the middle elevation of the pre-supported region. When the excavation proceeds through the pre-supported region, the displacement at pre-supported section D selectively increases at locations of 1 to 2 m, 4 to 5 m, and 7 to 8 m from the cavern wall; this corresponds to the locations of continuous joints a, β , and γ The displacement values at section D are larger than those at post-supported section K. It is thought that continuous joints a affect the rock displacement at section D.

At the end of the excavation of the cavern, the displacement near the cavern wall at section D is slightly larger than that at section K. As mentioned in Section 4.1, denser joints and clear continuous joints are distributed in section D, while there are no continuous joints in section K. In spite of these worse rock conditions, the displacement at pre-supported section D is almost the same as that at post-supported section K. It means that preset cable bolts are effective as rock supports.

4.3 Axial force of the cable bolts

The axial forces of the L2 cable bolts at EL 564 m and the L6 cable bolts at EL 558 m, which are monitored with SMART instruments, are shown in Figure 19 for each excavated lift. The axial force occurs and increases from Lift 8 to Lift 11 as well as the displacement of the rock. The locations of the incremental forces coincide with the locations of continuous joints α , β , and γ In addition, it seems that the lower the lift is excavated, the deeper the incremental axial forces occur. This means that the rock responses against the cavern excavation are subject to joint movements.

If the preset cable bolts can control the movements of the joints, they can therefore contribute to the stability of the cavern and reduce the number of post-set supports required to secure the cavern stability.

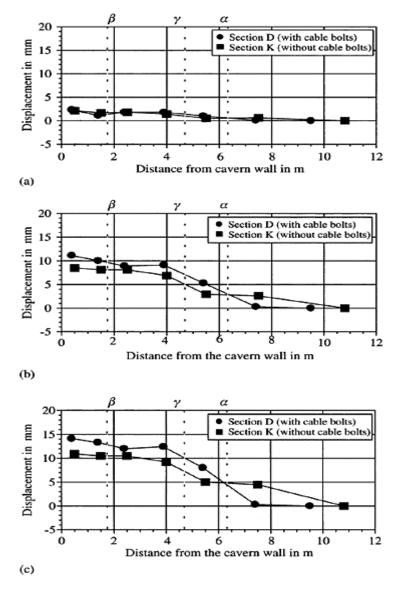


Figure 17. Rock displacement measured by the extensometers. (a) Lift 8 (EL560.1 m), (b) Lift 11 (EL552.2 m), (c) End of the excavation of the cavern (EL540.5 m).

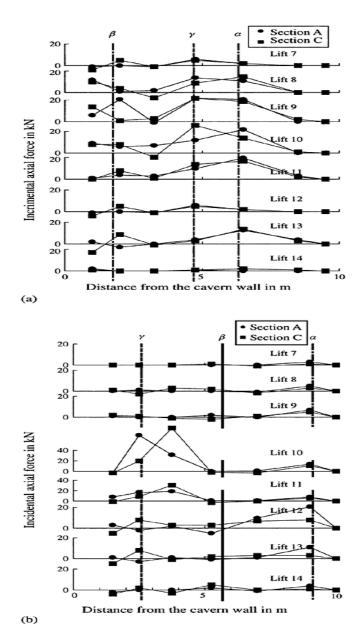


Figure 18. Axial force of the cable bolt due to the excavation of the cavern (a) L2 line, EL564 m, (b) L6 line, EL558 m.

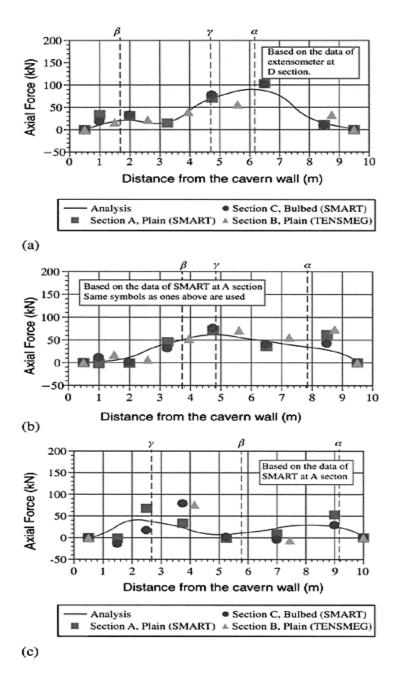


Figure 19. Axial forces of cable bolts (After the completion of the excavation.

4.4 Evaluation of the performance of the instruments for the axial force of cable bolts

The axial forces of the cable bolts are monitored by the three kinds of instruments during the excavation of the cavern. The terminated axial forces of L2, L4, and L6 cables after the completion of the excavation are shown in Figure 20. Outstanding forces are found at the locations corresponding to clear continuous joints α , β , and γ . Furthermore, a good agreement is indicated for the monitored forces by each instrument.

An analytical examination is made to verify the accuracy of the monitoring. Field experiments are simulated with the distinct element method (DEM) (UDEC Itasca) using the model shown in Figure 21, in which cable bolts are modeled by the spring (elastic modulus: 200 GPa) surrounded by the grout (elastic modulus: 8.8 GPa) combining the rock with the shear spring (shear modulus: 100 MN/m/m, shear strength: 0.4 MN/m). The mechanical parameters are set based on in-situ pull-out tests on the cable bolts. The analytical results preferably coincide with the monitored forces on cable bolts L2, L4, and L6.

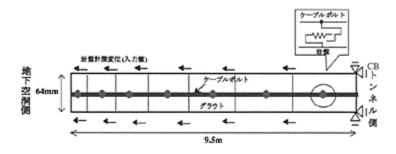


Figure 20. Mechanical model of cable bolts.

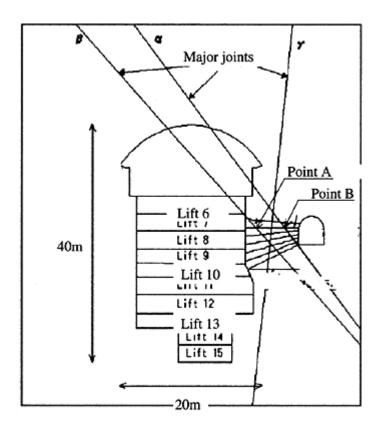


Figure 21. Section of the cavern for numerical analysis.

These results provide the evidence that each instrument is available for the monitoring of the axial forces of the cable bolts. It should be noted that TENSMEG and the strain gauge type involve some improper data, which demonstrate there are difficulties in the handling of these instruments. Meanwhile, SMART shows an excellent performance and is an effective measure for monitoring the axial forces of the cable bolts.

5 ANALYTICAL STUDY ON THE MECHANISM OF THE CABLEBOLT ACTION

Field experiments are simulated with the distinct element method (DEM) (UDEC Itasca) using the model shown in Figure 21; the model is 40 m high, 20 m wide, and has three continuous joints.

The parameters used in the simulation are identified in order to obtain the best match to the monitored displacement of the rock during the cavern excavation.

5.1 Rock displacement

The simulated displacement during the cavern excavation is shown in Figure 22 along with the monitored displacement. Both displacements show adequate conformity.

In the simulation, the displacement without cable bolts is evaluated and shows a distinct surplus from the wall to a depth of 2.5 m. It is presumed that the surplus displacement developed near the wall due to the response of joint β and that a reduction in the displacement is caused by the action of the preset cable bolts in the experiments. On the other hand, the preset cable bolts contribute to less of a reduction in the displacement in deeper areas of the rock.

5.2 Rock stress path and the axial force of the cable bolts

The rock stress levels at points A and B in Figure 21 are on the joint surface constructed with the preset cable bolts at EL 564 m. At nearby point A, the relaxation of the normal stress and the shear stress on the joint surface are better compromised in the case with

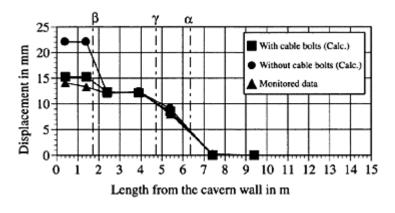


Figure 22. Displacement at EL 564 m.

cable bolts than in the case without cable bolts, as shown in Figure 23. The increase in the axial force of the cable bolts controls the decrease in the normal stress on the joints, as shown in Figure 24. For deeper point B, the difference in rock stress on the joint

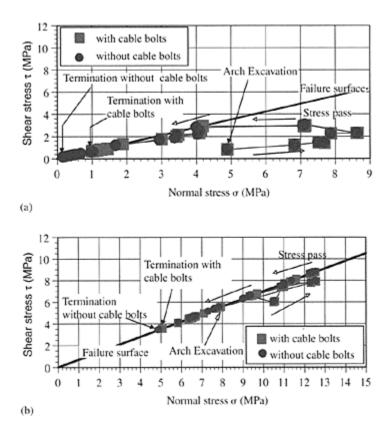
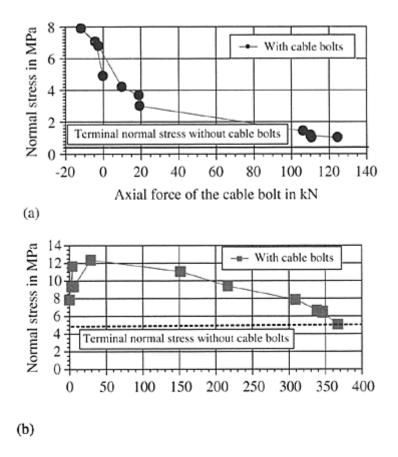
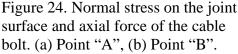


Figure 23. Stress pass on the joint surface. (a) Point "A", (b) Point "B"





surface is unclear between both cases, even though the axial force of the cable bolts is significantly higher than that for point A.

A similar response is obtained in the shear displacements shown in Figure 25. Namely, the shear displacement in the case with cable bolts is controlled at 10 mm compared with that in the case without cable bolts at point A, while there is little difference in shear displacements in both cases at point B.

At nearby point A, the decrease in the normal stress is controlled in the case with cable bolts and it must contribute to maintaining the shear strength of the joints and to reducing the displacement of the rock.

These responses show the typical advantages of preset cable bolts. Namely, preset cable bolts contribute to a reduction in the displacement and to an upgrade in the rock strength around the cavern while they work as fixed parts in deeper regions. Furthermore, the performance of the preset cable bolts may control the consequent deterioration of the rock caused by the excavation, which involves the joint opening and the redistribution of rock stress around the cavern, leading to rock deformation in deeper areas.

From the viewpoint of the cavern design, a quantitative evaluation of such advantages enables a reduction in the arrangement of such post-set supports as prestressed anchors, which require a longer period and a higher cost for their construction. In addition, preset supports can contribute to an improvement in the safety of the work and the avoidance of a delay in the

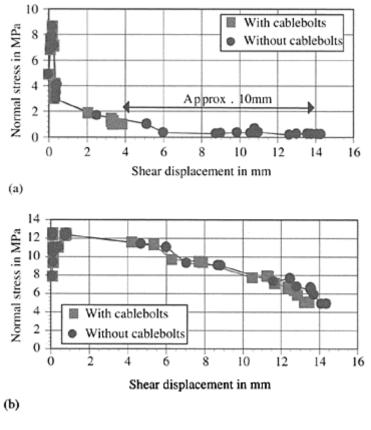


Figure 25. Normal stress and shear displacements on the joint surface. (a) Joint surface of "A", (b) Joint surface of "A".

scheduled construction period by preventing the accidental falling of surface rocks and/or the local failure of the rock due to the unexposed joints behind the cavern surface.

6 CONCLUSION

The construction of the Okutadami Expansion Hydropower Plant involved many technical difficulties caused partly by the limited construction period, the strict requirements of the preservation of the surrounding environment, and partly by the construction work adjacent to the operating plant. The successful excavation of the underground powerhouse cavern is presented here.

The stability of the cavern has been basically secured with the arch concrete and the post-set supports such as pattern rock bolts and the several prestressed anchors.

A study on preset cable bolting has also been made for the development of rational supports for future projects. Field experiments during the cavern excavation and numerical simulations have been conducted to clarify the effectiveness and the mechanism of the preset cable bolts. The following results have been derived from the study:

- 1. It has been experimentally confirmed that the rock displacement due to excavation can be controlled through the use of preset cable bolts. Since the displacement of rock occurs predominantly at the existing joints, preset cable bolts can help control the displacement of the joints.
- 2. A numerical analysis has successfully simulated the response of a cavern during its excavation involving rock deformations, rock stress levels, and axial forces of the preset cable bolts. Comparing cases with and without cable bolts, it is clear that the bolts work to control the joint displacement predominantly around the cavern.
- 3. Even though the effectiveness of preset cable bolts is limited to the area around a cavern, the bolts may alleviate the consequent deterioration of deeper rock. From an economic viewpoint, therefore, preset cable bolts can contribute to a reduction in the number of post-set supports necessary for the cavern and to the improvement of the cavern construction.
- 4. The application of SMART, the first of its kind in Japan, shows an excellent performance, and is an effective measure for monitoring the axial forces of cablebolts.

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Seismic and support behaviour, a case study: the April 22nd, 2003 Rockburst, Reservas Norte sector, El Teniente Mine, Codelco Chile

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ABSTRACT: On April 2003, the El Teniente Mine, particularly the "Reservas Norte" sector, was afectted by a 3.0 moment magnitude and high radiated energy $(3 \times 10^8 \text{ J})$ seismic event. Extensive damage (25-70 m) was generated affecting galleries all levels of the Reservas Norte sector. This paper presents the seismicity characteristics and the estimation of the peak particle velocity in the near field of the main event. This velocity field were correlated with the generated damage, considering the existing different support types and the geotechnical rock mass qualitity. As the main conclusion, in general terms the support systems worked in agreement with the support design under the dynamic stress generated by the seismic events. This kind of analysis will allow us to improve the design and the support characteristics.

1 INTRODUCTION

On April 2003, the El Teniente Mine, particularly the "Reservas Norte" sector, was affected by a 3.0 moment magnitude and high radiated energy $(3 \times 10^8 \text{ J})$ seismic event. Extensive damage (25–70 m) was generated affecting galleries all levels of the Reservas Norte sector.

The damage were classified from minor damage (unraveling rocks, joints, spalling) to major damage (violent rock overbreaking and floor breaking up) affecting the galleries from the undercut level Sub-6 (2120 m.a.s.l) to the Teniente 8 Level (1893 m.a.s.l).

2 PREVIOUS CONDITIONS TO THE ROCKBURST GENERATION

2.1 Mining method

Preundercut variant of the Panel Caving method is used in the Teniente Sub-6 Mine (West zone). To prevent the effects of the high abutment stresses associated with the conventional panel caving method was the main reason for introducing this variant. The preundercut method mitigates this effect by generating a distress volume below the undercut level.

From the construction point of view, the main difference between the pre-undercut and the conventional panel caving is the sequence of the gallery developments, the undercutting and the drawbell excavation (Cavieres and Rojas, 1993). As the name suggests, this variation of the panel caving method includes the excavation and blasting of the undercut level prior to the development and construction of the production level. In this way, three working zones are defined by this method:

- The undercut zone.

- The preparation zone in the production level (defined 22.5 m ahead of the undercut front).
- The production zone located some 45 m to 60 m behind the undercut front.

2.2 Conceptual framework for the induced seismicity in the Teniente Mine

In 1992, a global digital seismic network was installed, monitoring the induced seismic activity in the mine.

According to Dunlop and Gaete (1995), during 1992–1993 a conceptual framework was developed in order to relate the mining parameters to the rockmass seismic response characteristics. This relation would allow the control of the induced seismicity by means of the mining parameters modification.

A caving method is initiated by the blasting of the bottom volume of a rock mass column. The broken material is mined out creating cavities that allows gravity to continue the fracture process of the rockmass, producing new broken material.

The subsequent production generates that continuity of the breaking process, propagating the rockmass fractures to the upper levels. In general terms, the fractures correspond to the disruption of the structural pattern of the jointed rockmass.

A seismic event corresponds to the radiated energy associated with a rockmass rupture. Then, the induced seismicity is always associated to a rupture process affecting a competent rock mass.

The characteristics of the induced seismic events will be determined by the spatial and temporal distribution of the mining activities and conditioned by the geometrical, geological and structural characteristics of the mined rock mass.

The caving methods do not always allow an adequate control of rupture extension. According to the mining and the rock mass parameters, caving could generate large ruptures, i.e. high magnitute seismic events that can radiate enough energy to produce damage to the surrounding excavations.

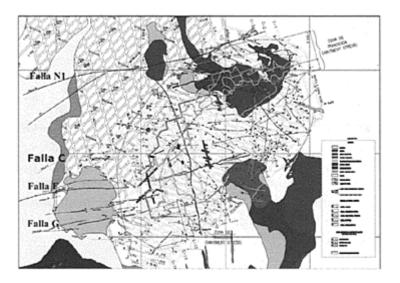


Figure 1. Geology and Faults in the Production level of Teniente Sub-6.

2.3 Geological, structural and geotechnical characterization of the affected sector

Some relevant geological structures are present in the Sub-6 production level. They include some faults like the C and G faults and some quartz dikes.

The dominant lithology is Andesite in a later hydrothermal ambient (HT), which has an uniaxial strength compression close to 118 MPa, an elasticity module equal to 35 GPa and a fair geotechnical quality according to the Laubscher's classification (1990).

2.4 Seismic event characteristics

The events generating the rockburst were clicked by the blasting of an undercutting level pillar using 800 Kg of explosive. The main event is a 3.1 moment magnitude event with a high radiated energy $(3 \times 10^8 \text{ J})$. Its focus is located almost 22 m above the Sub-6 production level, which was the level suffering the greatest damage. The rest of the event focus was distributed up to 130 m away from the focus of the main event (Figure 2).

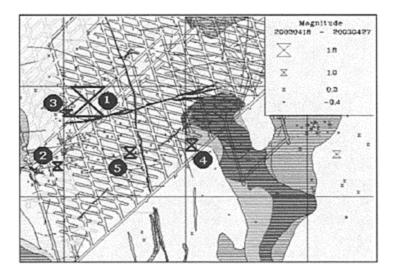


Figure 2. Plan view of the event focus locations numerated according to time sequence.

Characteristic		Event 1	Event 2	Event 3	Event 4	Event 5
Coordinates (m)	North	771,3	655,2	749,6	692,5	680,3
	West	841,0	790,8	806,9	1010,5	907,9
	High	2085,2	2099,9	2223,9	1991,5	1990,2
Hour (h)		16:49:21	16:52:02	17:43:02	20:13:25	20:23:24
Magnitude		3,0	1,1	0,8	1,6	1,6
Energy index		0,07	0,27	0,48	1,15	1,24
Apparent volume $\times 10^6$ (m ³)		789,8	6,2	2,1	37,8	3,7
Sismic moment (M ₀)×10 ⁹ (Nm)		35,5	55,2	21,8	32,8	37,0
Radiated energy (ER)×10 ⁵ (J)		2663, 1	0,8	0,4	47,3	61,1
Stations numbers		10	8	8	8	7
Energy relationed (S/P)	-		3,31	3,87	9,19	12,84

Table 1. Seismic parameters of the main events.

The principal characteristics of the generated events are showed in the Table 1.

3 ROCKBURST GENERATION MECHANISM

The low E_s/E_P parameter of the main events suggest a source mechanism including a shear mechanism in a fairly low confinement rock mass. These fractures could be generated by an instability condition of a large rock mass volume defined by a great distance between the extraction front and the undercutting front, and the column height of primary rock. This geometrical condition would create a large seismic active volume making possible a large rock mass rupture i.e. a large magnitude event (Figure 3).

3.1 Mechanical energy estimation

If we assume a 1% seismic efficiency for the main event the rupture total energy should be 100 times the radiated energy of the 3.1 magnitude event. The radiated energy has a value equal to 3.4×10^8 J (Table 1) then the total rupture energy should be close to 3.4×10^{10} J.

Considering the unstable rock mass showed in the Figure 5, three possible sources for this total energy could be the followings:

- A pure gravitational effect.
- A gravitational effect plus a confinement stress.
- A gravitational effect plus a confinement stress including the presence of some singularities in the induced stress field (i.e. abutment stress zone).

If a pure gravitational effect is considered, then an average movement of a few centimetres of the instable

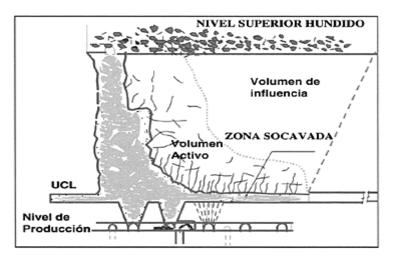


Figure 3. Diagram showing the active volume and the total influence volume for a preundercut Panel Caving variant.

rock mass volume would have created a total energy similar to the possible total energy associated to the rupture process of the main event, i.e. 3.4×10^{10} J.

A 10 cm average magnitude floor lifting was evidenced in the Ten-7 T/E level. In addition, some additional effects like sidewalls breaking, damage in the pillars, galleries sloughing, are present in the damaged levels.

If we add the work associated to the confinement stress to the pure gravitational effect, then the estimated mechanical energy is close to 9.5×10^{10} J.

Finally, if we consider that the mechanical energy is the result of gravitational effect plus the confinement stress under a singular condition (i.e. abutment stress), a 16×10^{10} J mechanical energy is estimated. This energy is 5 times greater than the estimated energy, 3.4×10^{10} J, dissipated during the rupture of the rock mass corresponding to the main seismic event.

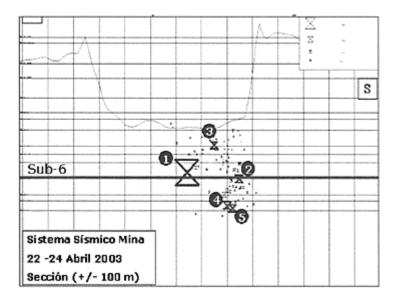


Figure 4. Section by the 700 N coordinate showing the seismic activity.



Figure 5. Unstable rock mass (yellow hatched) of approximately 30000 m^2 , limited by the C and G faults, the undercutting front and the extraction line.

Then, the total energy dissipated during the rupture of the rock mass including the main seismic event and all the damages can be easily explained by a gravitational effect (movement of an instable volume) plus the energy coming from the existing stress field including the abutment stress condition.

4 ROCK MASS AND SUPPORT DAMAGE MECHANISM

The damage mechanism can be associated to internal and external factors acting in the rock mass.

The internal factors included the presence of a degraded rock mass due to previous rockburst damages, a high excavation ratio in each level, the abutment stress presence, an incomplete and/or damaged support and an unfavorable structural condition associated to the "G" fault presence.

The external factor is the dynamic stress created by the seismic waves propagation. The Peak Particle



Figure 6. Severe damage in the production level (Sub-6).



Figure 7. Fair damage in the Teniente 7 standard level.

Velocities were estimated for the closest gallery walls with a maximum value of 0.8 m/s (Table 2).

The observed support damages were classified from light to severe damage. The damage description is as follows:

Severe damage: Severe rock unraveling that cover almost of 80% of the gallery section. Floor lifting greater or equal damage than 10 cm.

Fair damage: Minor rock unraveling that cover almost (50–80%) of the gallery section. Floor lifting close to 5 cm.

Light Damage: Light rock unraveling, minor spalling and breaking of side wall of galleries.

5 ESTIMATE CAUSE DAMAGE

In relation to the damage causes, either affecting the rock mass or the installed support, two assumptions are proposed:

- 1 According to the recorded seismicity, the rupture plane corresponding to the main event was estimated. The rock mass damages were associated to the induced PPV They were located close to the rupture plane and distributed around this plane reducing their intensity according to the distance from this plane.
- 2 The new empty room generated in the rock mass by the blasting of the undercutting level pillar originated a new induced stress distribution that could have created some unstable block conditions in the locations where the rocks mass suffered the greatest damage.

In both cases, it is necessary to include additional local conditions to explain the differences between observed and estimated damage. These additional effects could include removable rock mass block presence, missing and/or damaged support and a degradedrock mass due to previous rockburst damages. These possible conditions could explain the existing damage in galleries adjacent to undamaged galleries.

The Figure 9 shows the resulting damages that could be associated to the abutment stress presence where the rock mass suffered the increasing and rotational stress effects generated by the undercutting front (with the exception of the damage located in the Principal Haulage Level—Teniente 8).

Considering both assumptions, the peak particle velocity was estimated using the first one

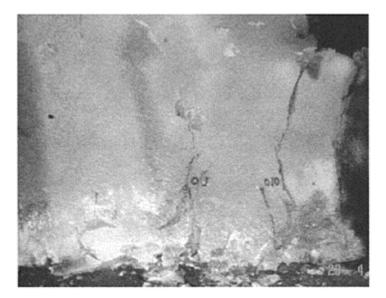


Figure 8. Light damage in the Teniente level 7 Haulage and extraction.

6 SUPPORT EVALUATION

The peak particle velocity at a distance less than 150 m from the event focus was estimated using the following relation (Mendecki, van Aswegen & Mountford, 1999): $RV_{MAX}=7\times10^{-9}\times(\Delta\sigma^2 M)^{1/3}$

(1)

Where:

R: distance from the source [m] V_{max}: peak particle velocity [m/s] σ: stress drop [N/m²] M: seismic moment [Nm] This relation was used due to the b

This relation was used due to the best fit between the estimated PPV and the observed PPV for distances between the event focus and the recording seismic station less than 150 m (Figure 10).

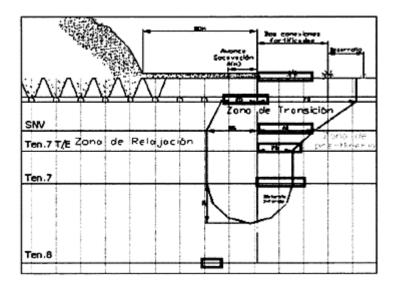


Figure 9. Damages observed in all the levels. They are located in the abutment stress with the exception of the Principal Level Haulage (Teniente 8) damages. The brackets indicate the damaged zones. Transversal view for the coordinate 700 N.

7 SEISMIC EVENT ENERGY GENERATED

The strain energy generated on the rockmass during a seismic event can be estimated according to the following relations.

 $E_{\text{kinetic}}=0.5 \text{ m}^*\text{v}^2$ (Kinetic energy) $E_{\text{potential}}=\text{m}^*\text{g}^*\text{def.}$ (Potential energy)

Where:

m: unstable rock mass condition mobilized according to the dynamic stress

v: particle velocity (m/s)

g: gravity acceleration=9,8 m/s²

def.: work length strain of support element= 0.8mm (=5mm*16%).

For the "def" parameter estimation only bolting anchor steel rebars (22 mm diameter) were considered excluding cable bolting. In addition, considering the observed steel rockbolts behavior during the rockburst, it is possible to assume that the rockbolts rebar strain is very small (almost 5 mm).

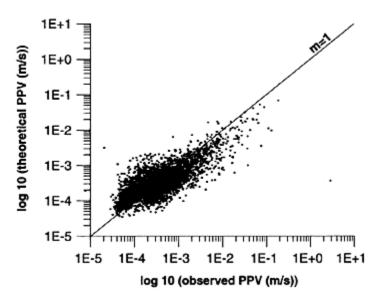


Figure 10. PPV estimated by Mendecki, A J., van Aswegen, G. & Mountford, P. (1999) considering the PPV observed at distances less than 150 m (March 2002-March 2004).

8 ENERGY SUPPORT INSTALLED ESTIMATION

The support system absorption energy necessary to equilibrate the rock mass generated energy was estimated according to the following relation:

E(support absorption)= $(2/3)*\sigma_r*\epsilon_r*v$

(2)

Where:

 σ_r =yield stress of the material ε_r =strain of the material=16%, according to the steel type of the rockbolt.

v=support volume including into the strain of the rock mass= $\pi \Phi^2 L/4$ L=free length of the support element=10 mm

 $\phi_{=\text{element diameter.}}$

Level	Damage type	Estimated PPV	Safety factor
Sub-6 production	Severe	0,445	0,66
	Fair	0,609	0,37
	Light	0,803	0,49
	Undamage	0,174	13,9
Sub-6 undercutting	Severe	-	_
	Fair	-	_
	Light	0,298	1,54*
	Undamage	0,195	3,16
Ventilation sub level	Severe	-	_
	Fair	0,441	0,75
	Light	_	-
	Undamage	0,113	6,08
Ten 7 T/E	Severe	-	_
	Fair	_	_
	Light	0,326	0,75
	Undamage	0,248	1,18
Ten 7 standard	Light	0,097	5,09*
	Undamage	0,176	2,74

Table 2. Safety factors for rockburst affected zones.

* No support and/or damaged, and a rock mass degraded due to previous rockbursts.

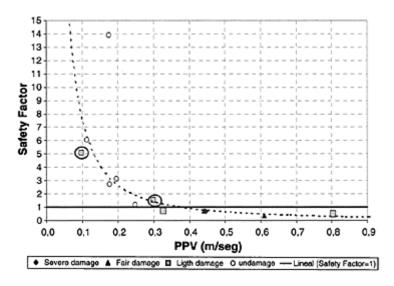


Figure 11. Safety factor estimated from PPV and installed support.

Table 2 shows the safety factors for each zone affected by the rockburst. They were defined as the energy absorbed by the support system to the energy generated in the rock mass quotient.

The damage areas include safety factors less than one. Although, there are some locations with light damage that probably present a safety factor greater than one due to some local conditions.

It is necessary to consider that particle velocities are only estimated and that each mining level has a particular condition in relation to the geology factors, gallery sizes and support conditions. These factors that should be considered for the evaluation process.

9 CONCLUSIONS

- The main cause of the seismic event generation corresponds to a great seismic active volume in an unstable condition due to a large distance between the extraction front and the undercutting front.
- The main rock mass and system support damaging mechanism is the dynamic stress induced by seismic event associated to an estimated PPV field with 0.8 m/s as the maximum value, the poor quality and incomplete condition of the installed support, the abutment stress presence and the cumulated damage of rock mass resulting from previous rockbursts affecting the same area.
- In general terms the support systems worked in agreement with the support design and the dynamic stress generated by the seismic events.

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Integrated ground support design in very weak ground at Cayeli Mine

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ABSTRACT: The Cayeli Copper-Zinc mine is a shallow underground

mine with very weak ground conditions. Although rehabilitation has always been an issue at the mine, the extent and degree of problems had increased in recent years as the extraction ratio has increased in the upper mine. An intensive ground instrumentation program was recently undertaken at the Cayeli Mine in order to rationalize ground support practices. This study has resulted in design of a new integrated support system and new ground support standards for the mine. This paper presents the results of this study and discusses the resulting new integrated support standards implemented at the mine.

1 INTRODUCTION

The Cayeli underground copper-zinc mine is a modern fully mechanized 3,500 tpd underground copper-zinc mine located in the Black Sea region of Northeast Turkey near the town of Cayeli (Figure 1). The mine is approximately 28 km east of Rize and 100 km west of the border with Georgia. The Cayeli mine site is located in the foothills of the Pontid mountain range. The mine is operated by Cayeli Bakir Isletmeleri AS (CBI), which is a Turkish company owned 55% by Inmet Mining and 45% by state-held Eti Holding AS and produces copper and zinc concentrates.

The ore is mined from a Volcanogenic Massive Sulfide (VMS) deposit, which is characterized by very weak host rock conditions. The deposit lies at depths varying from 40 m to 500 m below the portal entry elevation. The mining method employed is retreat transverse longhole open stoping with delayed backfill (Yumlu 2000). Primary and secondary stopes are 7 m wide, on average 25 m long and 25 m high. Backfill is necessary for pillar recovery and for the stability of the openings. Figure 2 shows a partial longitudinal section of the mine.



Figure 1. Location of Cayeli Mine.

Due to the very weak ground conditions, rehabilitation has always been a problem. During late 2001 and early 2002 an extensive underground field-testing and instrumentation program was conducted to rationalize ground support practices. Four instrumentation sites were established at Cayeli in the first quarter of 2002. The information gained from these test sites led to the design of a new integrated support system and new support standards for the mine. This paper presents the results of the test program and discusses the effectiveness of the new integrated support standards.

2 GEOLOGICAL CONDITIONS

The hangingwall lithologies observed at Cayeli, in which most of mine infrastructure is located, consist of alternating layers of basalt and tuff. The contacts between these rock units can be highly contorted and irregular. The basalt rock units vary from highly competent, blocky basalt to chloritic basalt. The degree of alteration in the chloritic basalt can also vary strongly and in some areas occurs as crushed, apparent shear zones within the blocky basalt. The highly chloritic basalt is extremely weak and is easily broken into fine fragments by a single blow from a geological hammer. The standup time for the chloritic basalt is extremely short.

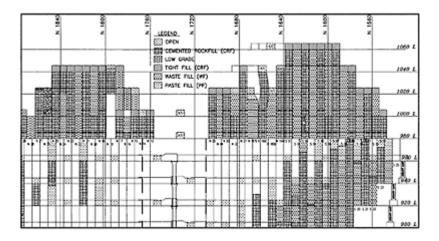


Figure 2. Partial longitudinal projection of the mine showing backfilling and mining method (looking east, not to scale).

Red and green tuff formations occur in the hangingwall. Both types of tuff can have strong clay alteration and are of similar geomechanical quality. When exposed the intact material can be cut with a knife and or easily broken with a single blow from a geological hammer. Initial standup time for the tuff is generally sufficient. If the tuff becomes wet however it usually degrades to a wet, plastic clay.

The footwall formations are Ryholite, which can be very weak or very strong depending on the degree of alteration. In the upper part of the mine the footwall rock located very close to the orebody was very weak; hence the major infrastructure was placed in the hangingwall. Further drilling and mining through the footwall below the 800 level elevation on mine grid (1000 mL on the mine grid corresponds to the sea level in actual) has revealed that at depth the footwall is better geomechanically than the hangingwall. In 2002 a decision was made to move the infrastructure below the 800 level into the footwall.

The massive sulfide formations are reasonably competent but much of the ore is very granular. The clastic and black ore are generally quite competent while the yellow and vein ore are very friable. These ore types break into fine pieces with only a few blows of a geological hammer.

3 FAR FIELD AND MINE INDUCED STRESS CONDITIONS

No in-situ stress measurement has yet been conducted at the mine. Due to the weak nature of the rock, conventional overcore in-situ stress measurements are not likely to be successful. However, a project is currently underway to measure the stresses using an alternate back analysis technique called 'under excavation technique', (Kaiser et al. 1990).

The portal elevation is at 1100 m elevation on mine grid and all existing mining is within 300 m below the portal elevation. The mountains rise steeply behind the portal, however, such that actual stress conditions underground will be slightly higher than values calculated based on a relative ground surface elevation of 1100 m. An evaluation of failure modes in development headings at the mine suggested that the maximum principal stress should be vertical (i.e. gravitational). Simple two-dimensional model back analyses of a limited number of these failures supported this conclusion (Bawden 2001). At this time the horizontal to vertical stress ratio is assumed to be 0.5. Subsequent instrumented cable bolt test programs confirmed this general failure mode, showing that failure initiates in the lower walls and, as the walls buckle and fail, loads arch up into the back. Given sufficient time and depth of wall failure, back failure will also occur. These analyses support the mine geometry stress driven failure model and were critical to the design of the new ground support standards developed for the mine (Bawden 2002).

4 GROUND SUPPORT AND GROUND CONTROL PROBLEMS

All of the infrastructure development for the main ore zone above the 800 level is located in the weak hangingwall formations. Stress driven floor heave and sidewall buckling are the predominant failure mode observed in weak hangingwall rock lithologies such as the chloritic basalt and red/green tuff. When initially developed, the primary support system in the hangingwall openings consisted of wall-to-wall 100 mm thick mesh or steel fibre reinforced shotcrete and 3.3 m long Super Swellex bolts installed at 1.5 m×1.5 m spacing. Cable bolts have been used only occasionally in the ramp. Although the majority of the hangingwall development openings were stable during the early years of mining, demand for ongoing rehabilitation in the hangingwall development has recently increased in response to the increased extraction ratios in the main ore zone above the 900 level.

Conditions in the main ramp vary dramatically. In some areas the ramp is in good condition and has never required any rehabilitation. In other areas however the ramp has suffered severe deterioration and has required multiple passes of rehabilitation. Deterioration in the ramp appears to occur as general closure. The closure is expressed as buckling and cracking of wall shotcrete, cracking and shear of back shotcrete and floor heave. In a few areas ramp ground conditions have been poor since the ramp was first driven. However, in most areas deterioration began some time after the ramp was driven and in many of these areas deterioration has exacerbated over time. Rehabilitation in the ramp has consisted of scaling loose rock and spraying additional fresh fibrecrete combined with re-bolting. Non-reinforced concrete is being poured for the floor of the ramp, both to provide a high quality tramming surface and to 'close the arch' with the shotcrete ground support.

Ore strike drives are generally developed in combined massive sulphide and hangingwall waste. Buckling failure is commonly observed on the hangingwall side of these access drives. Back wedge failures peaking against the hangingwall contact are also common. Original ground support consisted of Split Set and swellex bolts and fibrecrete. Cable bolts were used occasionally. A major problem in this development is that the friction stabilizer bolts corrode off in a matter of months leaving only the fibrecrete support. Prior to 2002, the main support systems used at the mine included:

- Use of 100 mm thick fibrecrete in waste rock development.
- Use of 2.4 m long Super Swellex bolts as a primary bolting in the waste rock development at 1.5 m \times 1.5 m spacing.
- Use of 2.4 m long Split Set bolts at 1.5 m×1.5 m spacing and steel mesh in the ore development
- Use of 9 m long plain single strand cable bolts used in long term development areas at 2.5 m spacing, but on an as-required basis rather than part of a standard support pattern.

Of the above support systems it was found that the Split Set bolts were rusting and failing within a few months (about 6 months) due to corrosive environment. Additionally, there was a question as to how much support the plain strand cable bolts were actually providing. The Super Swellex bolts were also a concern, primarily due to cost.

5 EVALUATION OF SUPPORT EFFICIENCY

In order to control deformations in the mine development to a more satisfactory degree an integrated support system, consisting of a tough surface retention system along with deep-seated high capacity anchorage, was developed. Prior to implementing the ground support design changes a test program was implemented to confirm that the proposed deep anchorage system would function as designed and that it would provide support capacity significantly greater than that available from conventional plain strand cables. Four test areas were instrumented in early 2002 with SMART bulb and SMART plain cables along with Multiple Position Borehole Extensometers (MPBX's). This allowed load distributions along the plain and bulb cables to be quantified and compared as the rock mass responded to mining induced stress changes. The behavior of the fibrecrete and the primary bolts was also observed and documented for all test areas.

6 DESCRIPTION OF SMART INSTRUMENTS AND DESIGN METHODOLOGY

Mine Design Technology (MDT) in Kingston, Ontario, Canada developed the SMART Instrumentation. Detailed description of the SMART concept is given by Bawden et al. (2004). The SMART cable bolt measures the displacement of up to six anchor points as the cable stretches in response to tensile loading. Since the relative displacements between anchor points are known through movement of wipers across linear potentiometers in the readout head, the amount of cable stretch may be calculated. By measuring the extension or stretch between two known locations along the cable, the strain may be calculated. Since the load deformation relationship of the cable is known (from laboratory testing), the average tensile load between the two anchor points may then be calculated from the measured cable strain. The platable variant of the SMART cable used in this study has a potentiometric based head small enough to be grouted into a 60 mm diameter borehole. SMART MPBX's, on the other hand, passively measure how the rock mass is moving, while the SMART cables measure how much the cable bolts are stretching in response to these deformations. The fully grouted cable bolts are installed as passive dowels. Tension develops in these tendons in response to the rock mass displacements.

7 TEST RESULTS IN ORE HANGINGWALL DRIVE

A test cable bolt section was installed in the 960 HW south ore drive. Both plain and bulb test sections were placed in close proximity in an area with no significant variation in ground conditions. The ground conditions in the test area may be simplified into two geotechnical zones: hangingwall and ore zones. Rock mass properties of both the hangingwall and ore zones were assessed to be 'weak rock' according to the Barton Q classification system. A 40 m section was cable bolted, 20 m with single plain strand cables and 20 m with bulb cables. Standard ground support had been comprised of 100 mm thick fibrecrete and 2.4 m long Split Set bolts on a 1.25 m by 1.25 m spacing. Ten (10) m long conventional plain strand cables were installed in the first half of the test section while the other half was supported using 10 m long bulb cables. It was decided to install 4×10 m long plain strand instrumented SMART cables and 4×10 m long instrumented SMART bulb cables in the centre of the respective test sections. This allowed load distributions along plain and bulb cables to be quantified and compared, as the rock mass responded to mining induced stress changes. When installed in this manner, the loads on the SMART cable will be the same as what would be experienced by an un-instrumented cable in the same location. Additionally in each test section a 13 m long MPBX was installed in order to monitor ground movement beyond the limit of the deep anchorage. In each test section SMART cables were installed at 10° , 15° , 45° and 90° from the horizontal, referred to as locations 1, 2, 3 and 4 respectively, (Figure 3) with only hole 4 being completely within the ore zone. The behavior of the fibrecrete and the primary bolts was also observed and documented for both test sections. All cable bolts including the SMART cables used were 15.2 mm diameter low relaxation 7-wire strand (ASTM A416) with yield strength of 225 kN. All of the cables were single strand, plated with 25 cm \times 25 cm plates to increase the effectiveness of the cable, and then tensioned to 50 kN.

7.1 Bulb cables

Figures 4 and 5 show 'displacement versus time' and 'load versus cable length' for the bulb cable in location 1,

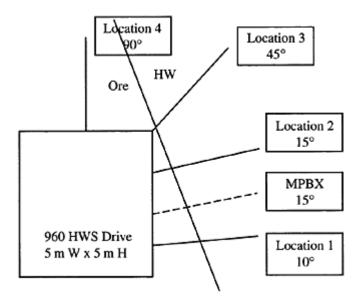


Figure 3. Cable layout for 960 HW south test section. Numbers refer to cable locations. View South (NTS).

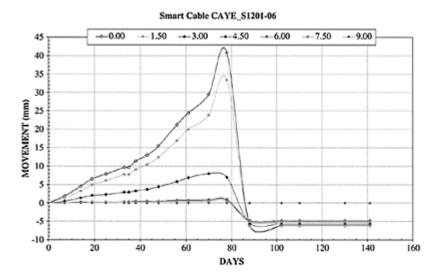
respectively. The bulb cable at this location has picked up load from the time of installation and was brought to rupture in 80 days. As shown in Figure 5 the cable load was concentrated between 1 and 3 metres into the wall and the cable did not experience load at a distance beyond 5 m from the wall surface of the 960 HW drive.

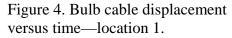
Figures 6 and 7 show similar plots for cable location 2. The interpretation for cable location 2 is the same as for cable location 1. Figures 8 and 9 show 'cable displacement versus time' and 'cable load versus distance', respectively, along the cable for cable location 3. Although this cable also began picking up load immediately upon installation, the rate of load accumulation was much slower than for cables 1 and 2 (i.e. $\sim 100 \text{ kN vs.}$ 255 kN at 80 days). Load on this cable also extends to a greater depth (>6 m) than for the cables in locations 1 and 2. Finally, Figure 10 shows 'cable displacement versus time' for the cable in location 4 (vertical in the back). As can be seen, no displacement (and hence no load) was picked up by this cable are very small and are distributed along the entire cable length.

7.2 Plain cables

The instrumented plain cable section was located about 20 metres north of the instrumented bulb section. The instrumented plain cables were installed in the same pattern shown in Figure 3 and results for this test section are given below. Figures 11 and 12 show 'cable displacement versus time' and 'cable load versus cable length'

respectively. At 80 days this cable had stretched a maximum of <35mm (vs. >40mm for the bulb cable). The plain cable continued to displace through the end of the reading period or 140 days. The largest





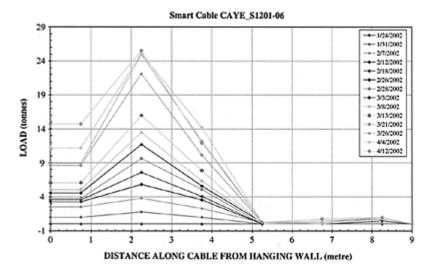


Figure 5. Bulb cable load versus distance—location 1.

cable loads in this case occurred between 1–4 metre depth, but the cable continued to displace and take some load to the toe of the cable at 9 m depth. Figures 13 and 14 show the same results for the cable at location 2. In this case at 80 days the cable indicated a maximum displacement of only 11 mm. Maximum cable loads are concentrated in the f irst 2 m of the cable, with cable loading continuing back to >7 m depth. Figures 15 and 16 show similar results for cable location 3. At 80 days this cable shows only about 20 mm displacement. Maximum cable displacements and load in this case occurs between 2 and 8 m depth and displacements continue beyond the end of the cable at 9 m depth from the wall of the drive. The vertical plain strand cable in the back did not take any load in the first 140 days of the test.

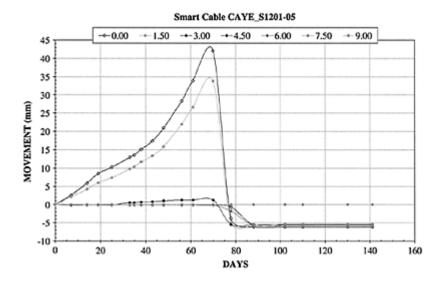
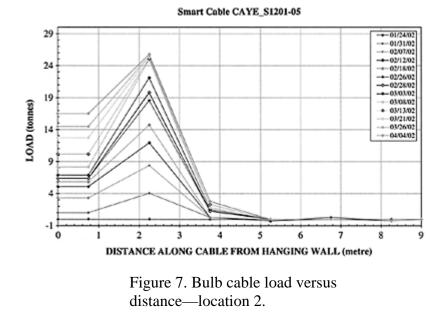


Figure 6. Bulb cable displacement versus time—location 2.



The MPBX installed in the bulb cable test section (Figure 17) showed perfect agreement with the surrounding bulb cables (Figures 4–10). The MPBX indicates ~43 mm of displacement of the wall at 80 days, with displacements concentrated in the first 3 m. The MPBX appears to have gone into shear at about 4 m beyond the wall. The MPBX in the plain cable section was damaged and did not provide useable data but visual data suggested greater movement than detected by the plain cables.

8 TEST RESULTS IN HANGINGWALL RAMP

A second test cable bolt section was installed in the 960–940 Main Ramp. A 40 m section was cable bolted, 20 m with 10 m long single plain strand cables and 20 m with 10 m long single strand bulb cables. 4×10 m long plain strand instrumented SMART cables and 4×10 m long instrumented SMART bulb cables were installed in the centre of the respective sections. In each test section a 1×13 m long MPBX was installed in order to monitor ground movement beyond the limit of the deep anchorage. The cable bolt installation in this area is as shown in Figure 18.

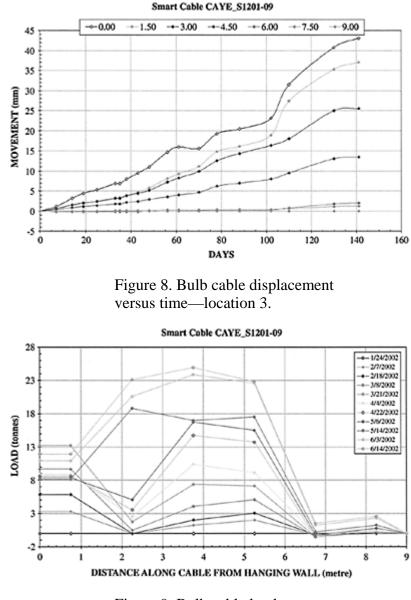
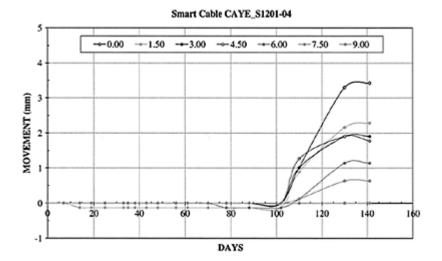
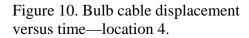


Figure 9. Bulb cable load versus distance—location 3.

These instruments were installed in early January and subsequently, on October 25, 2002, withstood a major ground failure associated with a suspected pillar failure about 50 m away in the ore. Figure 19 shows a photo of the wall of the ramp at the bulb cable test location. Note that, in this photo, the shotcrete shows no sign of damage or cracking. Figures 20 and 21 show 'displacement versus time' and 'load versus depth' respectively

for the lowest bulb cable located in the east ramp wall (i.e. facing the orebody). The data shows a clear acceleration in cable bolt stretch (and hence load) at the time of the October 25th failure, with movement concentrated at a depth of about 5 m into the wall. A plain strand cable test section was located immediately adjacent to the bulb cable test section shown in Figure 18. Figure 22 shows damaged shotcrete in the wall of the plain strand test section.





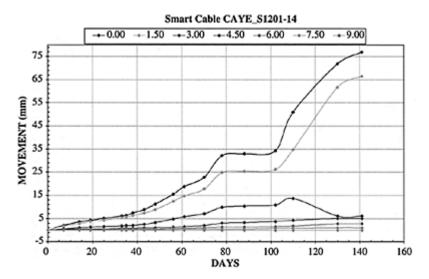


Figure 11. Plain cable displacement versus time—location 1.

Figures 23 and 24 show 'displacement versus time' and 'load versus depth', respectively, for the plain cable equivalent to the bulb cable shown in Figure 18. While both cables show similar accelerated movement after the October 25th event, the plain cable shows about 10 mm of additional movement compared to the bulb cable. Additionally, the maximum apparent strain with the plain strand cable occurs at the toe of the cable (Figure 23) versus between 4.5 and 6.0 m depth for the bulb cable (Figure 19). These results indicate that the stiffer bulb cables have constrained the development of the plastic damage zone to a 5 m depth into the wall of the ramp. The plain strand cables however allow larger wall movements that extend to greater depth into the ramp wall. These greater wall movements ultimately result in rupture and damage to the shotcrete lining as shown in Figure 22. Figure 25 shows a photo of much more extensive damage to the shotcrete lining in the 940–960 ramp immediately above the cable bolt test section where much less effective wall bolting had been done.

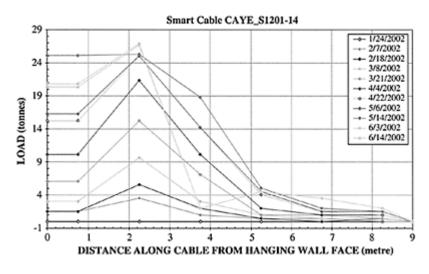


Figure 12. Plain cable load versus distance—location 1.

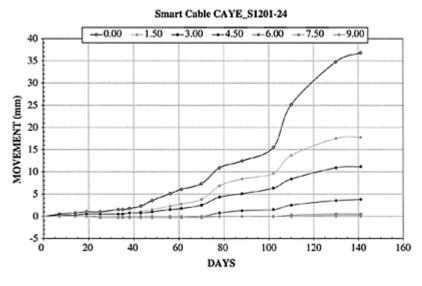


Figure 13. Plain cable displacement versus time—location 2.

9 DISCUSSION AND ANALYSIS OF RESULTS

Comparison of the output from the instrumented cables from the bulb versus the plain strand test sections indicate the following:

- the bulb cables in the walls loaded to peak capacity much more quickly than the plain strand cable (i.e. they are a stiffer support),
- plain strand cables in the walls eventually achieved full capacity but only after almost twice the time,
- the bulb cables contained deformation in the walls to the first $\pm 4m$ from the wall of the drive,
- the plain strand cables showed movements occurring beyond the total 9 m length of the support,
- both the bulb and plain back cables provided no useful function during the 140 days of monitoring,

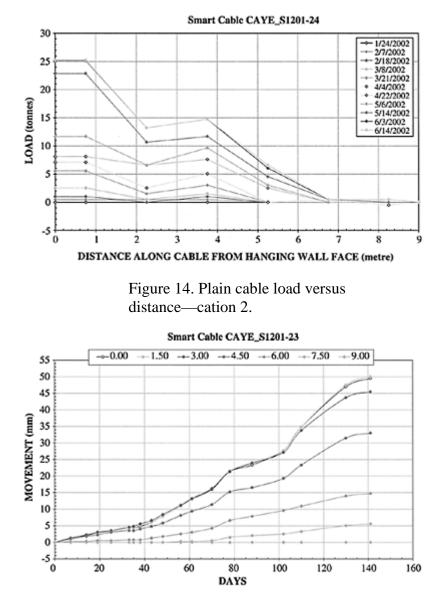


Figure 15. Plain cable displacement versus time—location 3.

- the bulb cables in locations 1 and 2 loaded at identical rates, while the cable at location 3 loaded more slowly, and the plain cable at location 1 loaded most quickly while cables at locations 2 and 3 loaded more slowly. These results have helped the mine develop a better understanding of how cable bolts behave in the weak ground conditions at the Cayeli mine. Single strand plain cable bolts are unable to resist the non-elastic stress driven deformations that occur at the mine. The basic function of the cables is to restrict growth of the plastic yield zone and to tie it back to the intact rock mass. Furthermore, this study has also shown that bulb cables with 0.5 m-bulb spacing are too stiff.

Visual observations confirmed that much more significant wall damage occurred in the plain cable part of the 960 HW test section, requiring about 1 m of scaling, while almost no scaling was done in the bulb cable test section. The combined data indicate that the

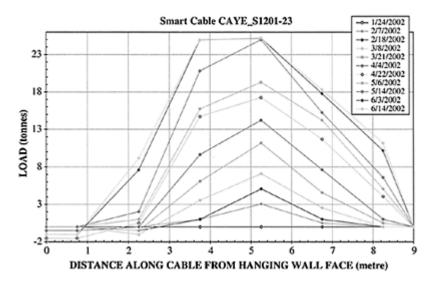
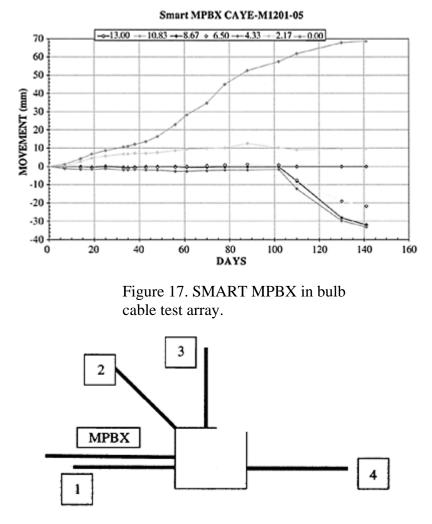
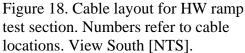


Figure 16. Plain cable load versus distance—location 3.





stiffer bulb cables help to control the depth of the developing plastic zone in the weak hangingwall lithologies. This, in turn, provides increased confinement on the rock behind the plastic zone increasing its strength and also helps to control the total amount of bulking in the wall. The plain cables on the other hand allow much more deep-seated movement and the resulting development of a much deeper plastic zone. The interpretation is that the increased bulking associated with the deeper plastic zone associated with the plain strand cables results in more extensive damage to the shotcrete liner support requiring more extensive scaling and rehabilitation.

Another major factor indicated by the 960 HW section is that the failure, in this area at least, begins from the lower corner of the drift on the hangingwall side and migrates up toward the back. In this test the installed back cables took no load in the first 140 days. However, significant back failures have occurred in

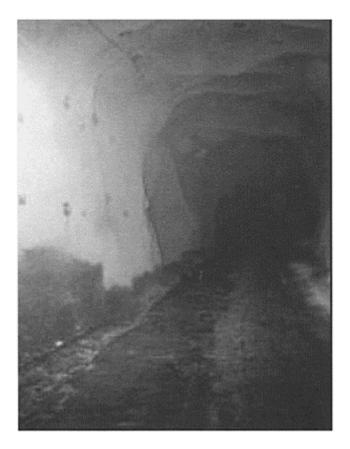


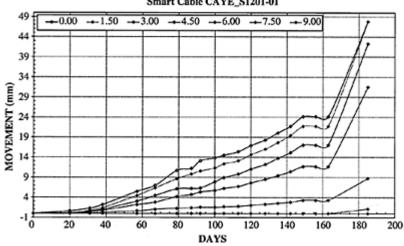
Figure 19. Wall damage in instrumented bulb cable section.

the ore hangingwall drives in the past. These may be progressive failures, where wall failure such as described above result in multiple passes of wall scaling and rehabilitation. This progressive wall failure results in an ever-deepening plastic zone in the hanging wall, which ultimately widens the effective back, finally resulting in back collapse. Hence, the fact that the back cables did not take load in this test does not necessarily mean that these cables should be discarded.

While these tests were ongoing the mine evaluated alternate primary bolt systems. Split Set and swellex bolts were eliminated from use in the long-term development, such as the main ramp and the hangingwall access drives in the massive sulphide, due to the severe corrosion problems. A decision was made to switch to use of resin grouted rebar in the development of long term openings and cement grouted rebar in rehabilitation of long term openings. Use of Split Sets is strictly limited to development of stope over cut and undercut sill drifts where the corrosion of Split Sets is not a major concern as they are usually turned into stopes, mined and filled in less than 3 months. Super Swellex bolts were totally eliminated from the mine due to their corrosion problem and high cost.

10 NEW SUPPORT STANDARDS

The results discussed above have shown that significant plastic yield zones develop around the opening. These are attributed to the mining induced stress and that the primary bolts were not long enough to pass through the plastic yield zone around mine openings. In order to control deformations in the mine development to a



Smart Cable CAYE_S1201-01

Figure 20. Bulb cable displacement versus time—location 1.

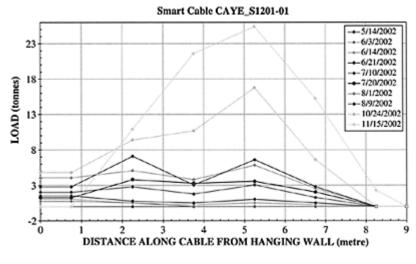


Figure 21. Bulb cable load versus depth—location 1.

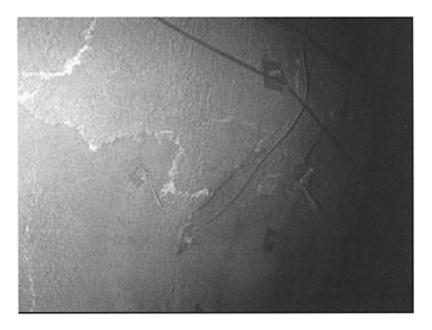


Figure 22. Wall damage in instrumented plain cable section.

more satisfactory degree an integrated support system was designed to have the following properties; (i) a tough surface retention system consisting of a full ring fibrecrete support

combined with 2.4 m long resin or cement grouted rebar at $1.0 \text{ m} \times 1.0 \text{ m}$ spacing and (ii) deep-seated high capacity anchorage system consisting of long twin strand bulb cable bolts, having a 1 m bulb spacing, preferably anchored beyond any plastic yield zone surrounding the opening and plated against the surface of the fibrecrete.

Figure 26 shows the new support system developed for the Cayeli mine.

11 CONCLUSIONS

Instrumented SMART cable bolts and SMART multipoint borehole extensometers (MPBX's) provided background information as to how the rock mass behaves. This information was then used in the development and

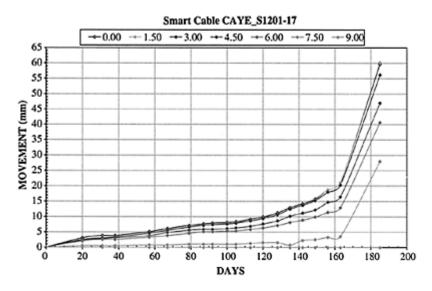


Figure 23. Plain cable displacement versus time—location 1.

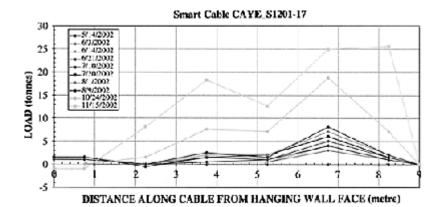


Figure 24. Plain cable load versus depth—location 1.



Figure 25. Picture showing damage to shotcrete beyond cable test section.

implementation of an integrated support system, consisting of a tough surface retention system combined with a deep-seated, high capacity anchorage system for use throughout the mine. The test results have led to the adaptation of the following ground support standards:

- Conversion from plain-strand cable bolts to bulb cable bolts to provide better bonding and loadcarrying capacity.
- Use of double-strand cable bolts rather than single strand to increase the ground support capacity.
- Integration of cable bolting into the standard drift primary support pattern.
- Modification of the cable bolting pattern by installing rebar bolts and cable bolts right down to the floor of the drift on each side wall.
- Minimizing the use of Split Set bolts by limiting their use to sill drifts in ore where only very shortterm use will be required.

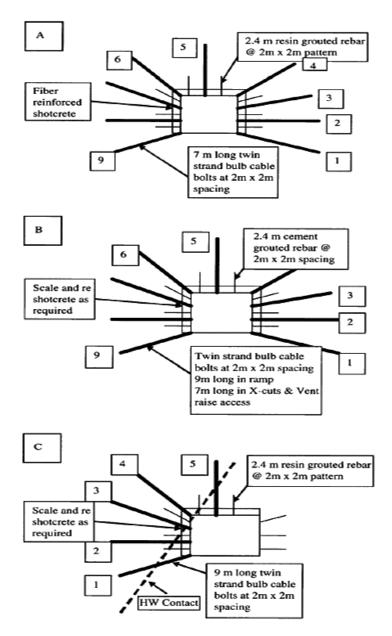


Figure 26. (a) Waste development rehabilitation standard (b) New waste development support standard (c) Ore HW access drift support rehabilitation and new. Using resin or cement grouted rebar bolting as a primary means of support in major long-term ore and waste development drifts and in rehab areas.

Mine Engineering is continuing to focus on the installation of SMART instrumentation throughout the mine. The results obtained from this ongoing instrumentation and monitoring program will result in further changes to the support standards as the mining and footwall development activities move further and deeper in the ore body.

Since it is extremely unlikely that continuing movements can be completely stopped in the very weak hangingwall formations, ground support is being instrumented in critical areas of the mine in order to ensure timely replacement of cable bolt support and to minimize further damage to this infrastructure.

ACKNOWLEDGEMENTS

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2 Rock mass characterisation

Three-dimensional rock mass characterisation for the design of excavations and estimation of ground support requirements

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ABSTRACT: Rock mass characterisation is a fundamental aspect of excavation and ground support design. Limitations of traditional geotechnical domaining may include the inability to readily identify variations in local rock mass conditions which, in some circumstances, may lead to sub-optimal reinforcement and ground support schemes being adopted. A method has been developed to construct 3-dimensional block models that account for local variations in rock mass conditions and that can be used to develop site specific excavation dimensions and estimate local rock reinforcement and ground support requirements. Three-dimensional rock mass model construction methods and considerations are described and examples of some results are presented.

1 INTRODUCTION

Rock mass characterisation is a fundamental aspect of excavation and ground support design. Traditional rock mass characterisation methodologies endeavour to divide the rock mass into domains of similar geotechnical characteristics and to report the likely range of rock mass and mining conditions expected to be encountered within each domain. Limitations in the ability of these traditional methods to visualise where local variations in rock mass conditions exist may lead to conservative designs and potential economic risks to projects.

The 3-dimensional model of rock mass conditions can be used to develop site specific excavation dimensions and to estimate local rock reinforcement and ground support requirements, using established empirical methodologies. The models can be used to optimise excavation and ground support design, potentially reducing economic and operational risks. The models can also be used as an ongoing management tool to record observed rock mass behaviour and ground support performance.

2 MODELLING FUNDAMENTALS

The main purpose of a 3-dimensional geotechnical model is to accurately model the subtle variations in geotechnical parameters within the rock mass. The secondary purpose of the model is then to assist the geotechnical engineer in the design of excavations and/or rock reinforcement, given these variations in the rock mass.

Before embarking on constructing a 3-dimensional model, a number of fundamental questions need to be considered.

2.1 Input data quality, quantity and distribution

Firstly, all models are constructed using data as input (i.e. the factual basis for the model). The input data, therefore, must be of sufficient quantity and quality such that it leads to the desired accuracy and reliability of the end result.

2.1.1 Data sources

The source and nature of the input data will have a direct impact on data accuracy and reliability. Care must be taken to understand and appreciate the different reliability and accuracies of each of the various data sources used to represent geotechnical input parameters. For example, a variety of data sources may be used to represent intact rock strength ranging from field index estimates to point load tests to unconfined compressive strength tests. Each of these sampling/test methods have different reliabilities and accuracies and the use of each data source must be accounted for within the model.

2.1.2 Data distribution

Data distribution concerns the availability and 3-dimensional location of all input data types. When assessing the distribution of data, one must firstly assess whether there is sufficient geotechnical information (i.e. each parameter) available for all geotechnical "domains" under consideration. Secondly, it is important to assess the 3-dimensional distribution of that geotechnical information (i.e. is it concentrated in one corner/area of the "domain" or distributed evenly?).

2.1.3 Data deficiencies

Where it has been ascertained that geotechnical information is limited or deficient in certain geotechnical "domains" or areas, contingencies may need to be devised. Strategies may include;

- utilising different modelling techniques for various sections of the rock mass (i.e. that are applicable and supportable to the amount/type of input data available),
- supplementing the input data by collection of additional data (if budget/time constraints permit).

2.2 Modelling methodology and accuracy

Once a model has been created, the accuracy of the model needs to be ascertained, regardless of the accuracy or adequacy of the input parameters. The model must be able to accurately predict the full range of anticipated conditions for each parameter within each domain. In assessing the accuracy and reliability of a model, it is worthwhile asking the following questions;

- Is the chosen modelling technique supportable with the amount and distribution of input data available?
- Have the correct modelling techniques been used for each domain?
- What is the accuracy of the model compared with reality?
- If the model is being used for a rock engineering process, can the model be calibrated using empirical data?

2.3 Objective and/or end-use of the model

As discussed earlier, the principle objective of a 3-dimensional geotechnical model is to represent the variability of geotechnical parameters in the rock mass. A secondary use is to assist in engineering design. In this regard, once a model is created, it must be ascertained whether the model can be used for its end purpose;

- Is the level of precision and/or accuracy of the model acceptable to the end user?
- Is there any engineering design process that is particularly sensitive to variations in parameters or characteristics of the model?

3 GEOTECHNICAL DATA COLLECTION AND DATABASES

As discussed earlier, the generation of 3-dimensional rock mass models is only supportable where there is a sufficient quantity and quality of detailed geotechnical data available. A growing trend has recently been observed within the Australian mining industry of systematic collection, storage and manipulation of geotechnical data. This growing trend of high quality and quantity data sets may now enable construction of sufficiently robust 3-dimensional rock mass models.

The format of geotechnical information is critical to ensure successful inclusion into electronic databases and for subsequent use in geotechnical modelling.

When designing a geotechnical database, standard fields, codes and reference tables need to be defined for both rock mass and discontinuity data. This is to ensure that data values entered are consistent, and unambiguous. This is also important when undertaking quantitative and qualitative statistical analysis of geotechnical parameters, and to ensure error-free execution of any computer codes and algorithms undertaken on this data within the database. Examples of code and algorithms may include;

- output of specific geotechnical parameters from the database to other electronic formats, for use in proprietary geotechnical software,
- calculation of rock mass classifications, and
- reporting and statistical functions.

3.1 Rock mass classiflcations

Rock mass classifications are an integral part of the modern geotechnical engineering discipline and play an important role, from initial rock mass characterisation studies to assisting in the design of detailed rock reinforcement and ground support schemes. The geotechnical engineer may need to consider a number of rock mass classification systems, depending on the engineering design task at hand. For example, Laubscher's MRMR system (Laubscher, 1990) would perhaps be preferred in initial assessments of caving, whereas the NGI-Q System (Barton et al, 1974) may be more appropriate in the selection of rock reinforcement or as input data for open stope design methods, such as the Mathews/Potvin Stability Graph Method (Mathews et al, 1981 and Potvin, 1988). Hoek et al (1995) have suggested that, when using rock mass classifications, it is good engineering practice to consider utilising a number of systems for verification.

The format of the collected geotechnical data, therefore, must allow flexibility such that the geotechnical engineer can utilise any number of rock mass classification systems or rock mass characterisation techniques as required. To this end, it is more preferable to record the "engineering geology" data, rather than interpreted rock mass classification parameters. For example, it is preferable to record planarity and roughness instead of an interpreted "Jr" from the NGI-Q system (Barton et al, 1974). Recording engineering geology characteristics as rock mass classification parameters will introduce bias, as these parameters are *interpretations*. In most cases, these parameters are also *simplifications* and, as such, information about the characteristics of the rock mass could be lost forever. In addition, it is also difficult to translate some parameters from one classification system to another (due to compatability issues) and also makes it difficult to audit the correctness of the rock mass model from this type of "data".

4 MINING SOFTWARE

In the design of excavations and reinforcement, variations in geotechnical properties need to be displayed along side proposed mining layouts. Contemporary mining software can now allow geotechnical data to be displayed along side proposed mining layouts in a variety of formats. Using some of the traditional geological tools within mining software, raw data from boreholes, and/or mapping traverses, can be displayed in 2 or 3 dimensions, with the ability to generate plans and sections of both rock mass and structural data.

The majority of mining software systems available in Australia (i.e. Surpac, Datamine, MineSight, Vulcan, Gemcom) can now access geotechnical data, via bi-directional links to ODBC-type databases. By exploiting these technologies, it is now possible to view geotechnical data and produce 3-dimensional block models using traditional "geological" and resource tools. However, some skill with these tools is still required. By using some of the automation features of mining software, together with the skills of an "expert" user, it is possible for geotechnical engineers to assist in development of automated macros to manage routine tasks, such as;

- 3-dimensional displaying of geotechnical parameters on-screen (ie from boreholes and mapping faces),
- 3-dimensional modelling of major structures,
- generating level plans and sections of parameters for plotting,
- generating plans and sections of structural data (including structural symbols),
- generation of grid and block models of geotechnical data.

5 MAJOR GEOLOGICAL FEATURES

Major geological features, such as major geological contacts, faults and folds, can have a significant impact on the design of excavations and rock reinforcement.

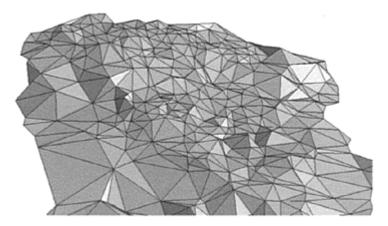


Figure 1. Example of a digitally modelled fault surface.

With appropriate geological data, major geological features can be modelled in 3dimensions as digital surface or solid triangulations using mining software. Figure 1 shows a fault modelled by generating a triangulated network, with the nodes represented by the interpreted down-hole intercepts with the fault. These geological features can then be displayed together with the proposed mining layout. The 3-dimensional surface/solids models can identify areas where additional reinforcement may be required, or where the design may need to be modified to avoid intersecting these potentially unfavourable features.

The presence of major geological features within the rock mass influences geotechnical characteristics of the rock mass, for example, a nearby fault may locally

increase the degree of fracturing (i.e. fracture frequency), increase weathering and decrease intact rock strength, and change discontinuity characteristics.

As major geological features can influence the engineering properties of a rock mass, 3-dimensional models of these features have become important tools in assisting in delineating, or *domaining*, the rock mass into broad areas of similar rock mass and/or structural characteristics.

6 THREE-DIMENSIONAL MODELLING TECHNIQUES

Essentially, geotechnical parameters can be described as "numeric variables" and, as such, the process for generating a 3-dimensional rock mass model is almost identical to the process used in geological and resource modelling.

The objective of 3-dimensional modelling is to try and simulate an entire area or volume of a rock mass from a limited number of sample points. In this regard, most models consist of a series of 3-dimensional lattice points. Each point within the model has the following attributes;

- Cartesian coordinate (i.e. x, y, z position relative to mine coordinate system).

- Parameter fields of interest (e.g. RQD, UCS, Fracture Frequency, etc.).

The main steps in the modelling process are summarised below;

- Evaluation of input data sources, data accuracy and reliability and data distribution
- Preliminary geotechnical domain definition
- Determination of the most appropriate modelling types for each domain
- Compositing input parameters into regularly sized data intervals
- Statistical analysis and sub/re-domaining (if required)
- Defining and applying interpolation techniques
- Model verification

6.1 Domain definition

Geotechnical domain definition is perhaps one of the most important aspects of 3dimensional modelling, regardless of estimation/interpolation method chosen. The main objective of domain definition is to divide the rock mass into areas, or volumes, of similar geotechnical characteristics.

A good understanding of the geological environment and geotechnical setting is imperative in determining which areas will contain similar rock mass characteristics and that will behave in a similar fashion. There are no standard procedures for the division of a rock mass into domains and this process largely depends on a good deal of experience and "engineering judgment".

The main criteria for geotechnical domaining should at least include;

- Weathering boundaries,

- Lithological boundaries,
- Intact rock strength variations,
- Variations in fracture intensity,

- Discontinuity set model and characteristics,
- Fault boundaries,

Figure 2 represents an idealised rock mass showing the influence of the various rock mass characteristics on the definition of potential domains (designated by the three letter codes in parentheses).

Development of a detailed geotechnical model may sometimes be hampered by geological complexity, and to a lesser extent, by insufficient geological understanding to adequately define detailed geological and weathering boundaries. Therefore, due to this geological complexity, it may be necessary to simplify the rock mass by generating 3-dimensional geotechnical domains only based on areas with similar rock mass characteristics.

In other circumstances, it may be necessary to simplify domains by dividing the rock mass into areas that

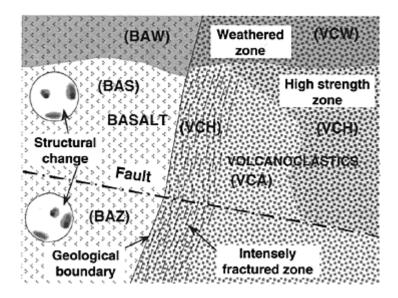


Figure 2. Schematic representation of possible criteria geotechnical domain definition.

will simulate ground conditions to be encountered during excavation. For example, these "domains" may correspond to the immediate mining hanging-wall, ore zone and immediate mining footwall.

Once domains have been decided, the raw geotechnical data from the database can then be assimilated into the 3-dimensional domains. This is usually done by "tagging" all geotechnical parameters with a domain code. In this way, all data can be sorted and statistically analysed by domain. This can provide the geotechnical engineer with a complete set of statistically analysed geotechnical design parameters for each domain, ready for use in, for example, probabilistic stability calculations, etc.

6.2 Model types

A number of model types have been developed in the geology discipline that are equally applicable to the geotechnical discipline. The main types are;

- Polygonal model
- Grid model
- Column model
- Block Model

For those not familiar with geological and resource modelling, the following sections are intended to provide a brief outline of the types of models typically used in geological and resource modelling.

6.2.1 Polygonal model

The polygonal model, as shown in Figure 3, is perhaps the most basic form of model. This model essentially consists of a polygon centred on a data point, with its boundary described equi-distantly between neighbouring data points. The value of each data point are assumed to apply equally to the entire area or volume of the polygon.

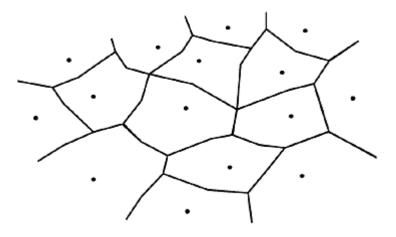


Figure 3. Example of a polygonal model showing data points and polygons.

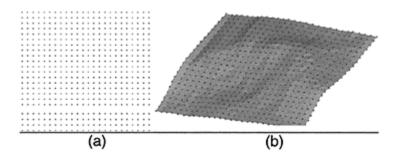


Figure 4. Example of a grid model showing equally spaced grid estimation points (a) shown in plan and (b) shown in perspective "overlain" on a fault surface.

6.2.2 Grid model

A grid model, as shown in Figure 4, is based on a series of equally spaced grid points. The grid can be "overlain" or "projected" onto a 3-dimensional surface, providing a true 3-dimensional model. The value of each grid point can be determined by a variety of methods, including interpolating values from nearby drill holes. These models are generally developed to represent surfaces (i.e. faults, geological boundaries, hangingwall/footwall contacts, etc).

6.2.3 Column model

The column or "reef" model, as shown in Figure 5, consists of a series of regular square shaped columns (i.e. in two dimensions) which have their top and bottom extents truncated or bounded by a surface.

6.2.4 Block model

Block models are perhaps the most sophisticated model type. Block models consist, as the name suggests, of a series of blocks or cells. Each cell has a centroid and extends in 3-dimensions to form a volume. Cells within a block model can consist of regularly sized cells (i.e. all the same volume) or may be divided into sub-cells. Sub-celling is a technique used to define resolution around complex shapes and close to boundaries. The type and amount of sub-celling allowed within the block model can be controlled and is usually dependant on the type of mining software used.

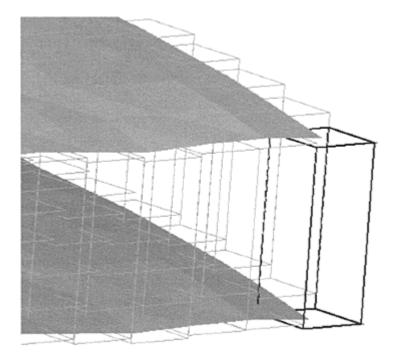


Figure 5. Example of a column or "reef" model, showing columns bounded by two 3-dimensional surfaces.

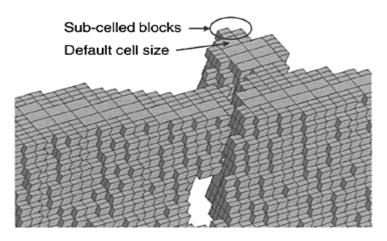


Figure 6. Example of a block model showing sub-celling.

6.3 Compositing

Compositing involves selecting an arbitrary sample size interval (e.g. 1 m) and splitting the original data length intervals into these new regularly sized intervals. Compositing requires weight averaging and combining of samples that cross the composite interval. Compositing ensures that the model receives the same volumetric weighting from each sample and that subsequent statistical methods provide unbiased results.

The composite interval size must be selected to reflect the "support" required for the model. "Support" refers to the "size and shape of the sample, the way in which it may have been taken and/or measured, and so on" (Clark, 1979). The choice of sample size will influence the result of any estimation process. For example, if samples are composited into large intervals, there is a possibility of the new composite intervals being "smoothed" or over-averaged. It is also important to ensure that compositing does not occur across geotechnical domain boundaries.

6.4 Geostatistics

In order to develop an appropriate and realistic 3-dimensional model of the rock mass, the fundamental geostatistical behaviour of rock mass and structural properties, for example intact rock strength, discontinuity continuity, and the management of the effects of drill hole bias need to be ascertained. Geostatistics can provide the necessary tools to understand the relationship between data values (i.e. spatial data) and the way in which these values vary in 3-dimensions. In geostatistics, one of the underlying assumptions is that the value of each data point is in some way related to its location (Matheron, 1971). The following sections only provide a brief outline of some basic principles of geostatistics and some of the more common estimation methods. For a more detailed and thorough introduction to geostatistics, readers should refer to Isaaks and Srivastava (1989).

6.4.1 Semi-variance and semi-variograms

The basic tool of geostatistics is semi-variance, which represents the measure of the degree of spatial dependence between samples. The "experimental" semi-variogram (i.e. actual sampled values) is a function of the semi-variance of samples with distance and is described in Equation 1.

$$\gamma^{*}(h) = \frac{1}{2n} \sum [g(x) - g(x+h)]^{2}$$
(1)

where $\gamma^*(h)$ =the experimental semi-variogram, g(x)=value at position x of first sample pair, g(x+h)=value at the position of second pair separated by distance h, and n=number of sample pairs (Clark, 1979).

Experimental semi-variograms can be geostatistically modelled by a semi-variogram function. This is perhaps analogous to fitting a normal distribution to sample data in classical statistics. A typical semivariogram model function is shown in Figure 7. It can be seen that the variance for samples that are close together (i.e. near the origin) is quite low, whereas samples further apart show more variance. This variance increases the

further samples are apart until it reaches some plateau (e.g. sill value) at a specific distance (e.g. range). After this range the data is no longer spatially related. At the origin there usually is some variance that cannot directly be attributed to the spatial

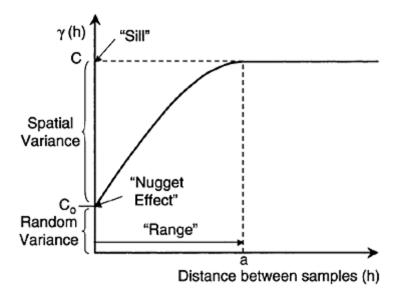


Figure 7. An idealised semi-variogram.

dependence and is related to the random variation of the sample values themselves. This random nature is called the "nugget effect" and may be attributed to a combination of errors in analysis and sampling, or due to the natural randomness of the variable under investigation.

There are a number of different variogram "models" available. The function shown in Figure 7 is the spherical function, which is described in Equation 2.

$$\gamma(h) = C_o + C \left[1.5 \frac{h}{a} - 0.5 \left(\frac{h}{a} \right)^3 \right]$$
when $h \le a$ (2)
$$\gamma(h) = C$$
when $h \ge a$

where $\gamma(h)$ =semi-variance, C_0 =nugget effect, C=sill, h=distance between pairs and a=range (Clark, 1979).

6.4.2 Estimation methods

A 3-dimensional model essentially contains a lattice of points (either regularly or irregularly spaced) in 3-d space. The values for each of these points need to be estimated from sample points (i.e. drill hole data, assay results, etc.). A number of estimation and interpolation methods are available, with various levels of sophistication. Some methods

of estimation and interpolation are briefly outlined below (listed from least to most sophisticated):

- assign value by constraint,
- nearest neighbours,
- inverse distance,
- ordinary kriging,
- indicator kriging.

6.4.2.1 Assign value by constraint

The simplest form of estimation is to assign values based upon what region or domain the estimating data point falls in. Generally this is done where the number of sample points is limited and is based on the results of classical statistics for points in that region (i.e. using the "mean" values of sample data points lying within that region/domain).

6.4.2.2 Nearest neighbours

Estimate by nearest neighbours essentially averages sample values within the immediate area of the lattice point. A "search ellipse" is also commonly used to control the amount and directional significance of samples (i.e. raw data points). A search ellipse limits the estimation and/or interpolation method to only those samples that fall within the ellipse. Search ellipsoids can be oriented, and have various aspect ratios to bias what samples are used in interpolation.

6.4.2.3 Inverse distance

The Inverse Distance (ID) and Inverse Distance Squared (ID2) interpolation methods are reasonably simple estimation methods that assume that the variance between values is based solely on the distance between data points, and nothing else (Clark, 1979). This method does not assume a detailed understanding of the statistical relationship between sample values and their variability in 3-dimensional space (i.e. variography). The method instead estimates values based on the weighted values of sample data points closest to the estimation point under consideration. The weighting is the inverse of the distance of the data point from the estimation point raised to a specified power (e.g. either 1 or 2).

6.4.2.4 Ordinary kriging

Ordinary kriging (OK), or "kriging" (after Krige, 1951), is an estimation method that provides weighting to sample points based on an optimal semi-variogram model. The estimate and estimation error (i.e. the error between the estimated value and the true value) will depend on the weights chosen. Ideally, kriging tries to choose the optimal weights that produce the minimum estimation error. In ordinary kriging, the estimation variance around estimation points is assumed to vary according to a normal distribution.

The methods used in kriging allow it to have an advantage over other estimation procedures in that the estimated values have a minimum error associated with them and this error is quantifiable. One of the properties of kriging is that it is an "exact interpolator"; that is, it estimates all the data (sample) points exactly (i.e. the variance between the estimate and the sample value is-zero at this location).

Quantifying the estimation error provides the engineer with a useful tool, in that it can be used to map the standard deviation of the estimate across the model. This can then be used as a "risk" map, to highlight size and extent of uncertainty in the model. These maps could be used, for example, to highlight where additional data may need to be collected to reduced uncertainty, or where more stringent management controls are required.

6.4.2.5 Indicator kriging

As indicator kriging (IK) is a fairly complex estimation method, only a brief introduction is provided. The main difference over ordinary kriging, is that this method does not assume normal distribution around estimation points. IK instead builds a cumulative distribution function (CDF) at each point based on the behaviour and correlation structure of "indicator transformed sample data points" in the neighbourhood (i.e. within the search ellipse). Indicator kriging works on a transform of the original data, whereby values are converted to either one or zero (i.e. 1 or 0) depending upon whether they are below or above a threshold or cut-off value. The kriging is then essentially ordinary kriging of these indicator values for each cut-off. The output is the probability of the point or block being estimated being above (or, by deduction, below) a particular cut-off value. Taken as a series, the indicators give the range of likely values for the block or point being estimated (i.e. provides a CDF for the estimation point/block).

Perhaps the greatest value of indicator kriging to geotechnical modelling is where it may be necessary to utilise a variety of data sources to model individual geotechnical parameters. For example, for a given geotechnical database, intact rock strength may be represented by field index estimates, point load tests, Schmidt Hammer hardness and UCS tests. The quantity and reliability of each of these "test" methods will vary across the study area, typically with data sets having a propensity of less reliable data over more accurate and precise data types (e.g. more field index tests than UCS tests). Each data set will also have its own unique probability distribution. Appropriate use of indicator kriging can allow for these various data sources to be successfully modelled.

7 MODELLING RESULTS

A number of examples of 3-dimensionally modelled rock masses are briefly presented, highlighting the potential uses of 3-dimensional rock mass models in rock mass characterisation and estimation of rock reinforcement and ground support requirements.

7.1 Rock mass characterisation using grid models

The following example shows how the use of a grid model can be an effective tool in visualising variations in rock mass conditions throughout the mine. A grid model was constructed for the hangingwall, footwall and mid-ore intersection of a longhole stoping operation in Western Australia.

Data from drill hole logging and underground mapping was combined into a database and used as the data source for estimating the various geotechnical parameters within the grid models. The data source was first composited into 1 m intervals. Only data relevant to each grid model surface, was used in the estimation process. For example, for the ore zone, only data samples lying within the ore zone were used. Similarly, for the footwall and hangingwall models, only data lying within a specified distance from the ore contacts was used. For example, data located in the footwall and lying within 5 m of the ore contact was used in construction of the footwall model. The interpolation method chosen for all parameters within this example was inverse distance squared.

The results of fracture frequency modelling for the footwall grid model surface are presented in Figure 8. The model presented highlights the variation in fracture frequency across the footwall surface. It can be seen from Figure 8 (Villaescusa, 2003), that a number of stopes are anticipated to be developed in very poor ground conditions, with potentially poor stoping performance predicted in these stopes. The traces of major structures are also shown, indicating the possible controls on high fracture frequency.

Apart from fracture frequency a number of other geotechnical parameters can be modelled, such as RQD, as well as defect characteristics. These parameters can be used to create basic models of rock mass classifications. In this example, the defect data was examined in order to generate models of estimated *Jn*, *Jr*, *Ja* and *Jw* (Barton et al, 1974) for each geotechnical domain.

The intact rock strength, together with the estimated insitu stress regime (σ_1 and σ_3 were estimated with depth) were used to estimate the SRF term in the NGI-Q system (Barton et al, 1974). All these estimated parameters (i.e. RQD, *Jn*, *Ja*, *Jr*, *Ja*, *Jw* and SRF) were then used to calculate the NGI-Q value for each grid model. An example contour plot of NGI-Q



Figure 8. Example of a contoured grid model of the footwall surface, showing values of fracture frequency, together

with stope outlines and traces of major structures (after Villaescusa, 2003).

values for the footwall grid model is shown in Figure 9 (Villaescusa, 2003).

7.2 Grid model and hangingwall cable support requirements

In the previous example, we saw that grid models can be used for basic rock mass characterisation and calculation of rock mass classifications. In the following example, a grid model is used to conduct empirical stope stability, including the estimation of hangingwall cable reinforcement.

The modified stability graph method (Potvin, 1988) is widely used in the mining industry to assist in the design of open stopes. The method has also been further developed by a number of authors (Potvin et al, 1989, Potvin & Milne, 1992, and Nickson, 1992) to estimate cable bolt reinforcement in stope design.

For the hangingwall grid model surface, the modified stability graph method was used initially to estimate the required hydraulic radius for each stope surface. Once the hydraulic radius was selected the cable reinforcement requirements could then be assessed.

The modified stability graph method relies on relating the modified stability number (N') to the hydraulic radius by way of a number of curves, each depicting various levels of stabilit+y. The modified stability number (N') is based on applying various factors to a modified version of the NGI-Q number;

N'=Q'*A*B*C

(3)

where A is a weighting factor related to the ratio of the Uniaxial Compressive Strength of intact rock to the induced compressive stress parallel to the stope surface under consideration, B is a weighting factor based on the orientation of the joint set that is most likely to detract from the stability of a particular stope surface, C is a factor that accounts for the influence of

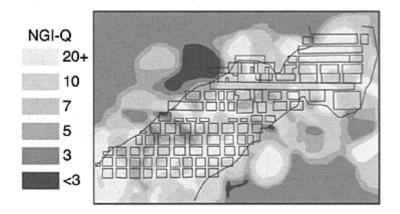


Figure 9. Example of a contoured grid model of the footwall surface showing NGI-Q values, together with stope outlines (after Villaesusa, 2003).

gravity on the stability of a stope surface, and Q' is a modified form of the NGI-Q system;

$$Q' = \frac{RQD}{Jn} * \frac{Jr}{Ja}$$
(4)

In order to calculate the modified stability number for the hangingwall surface, each of the constituent parameters needed to be estimated. In estimation of Factor A, a model of the anticipated induced stresses was also required. The induced stress at the mid-point of each hangingwall surface (based on the results of numerical modelling) was utilised to model induced stresses for the extraction sequence selected. This in turn was used, together with the intact rock strength model, to calculate the Factor A value across the hangingwall grid model surface. Factors B and C were calculated using a series of algorithms (based on the charts of Potvin, 1988) that used the dip and dip direction of the major structural orientations for each domain as input data, together with the variation of dip and dip direction of each stope surface. From this, the stability number N' could be calculated for each grid point, as shown in Figure 10 (Villaescusa et al, 2003).

Using Nickson's (1992) relationship between the modified stability number (N'), hydraulic radius (HR) and cable bolt density (cable bolts per square metre), it was then possible to estimate the required cable bolt reinforcement density for each of the hangingwall stope surfaces. The cable bolt densities for the hangingwall stopes are shown in Figure 11 (based on the stope outline shapes depicted).

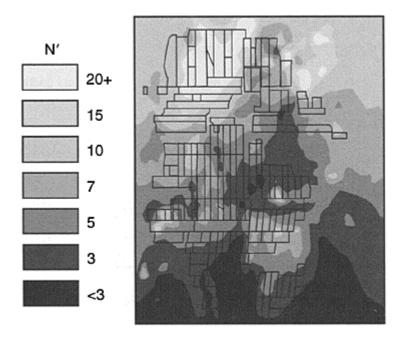


Figure 10. Example of a contoured grid model of the hangingwall surface, showing values of N', together with stope outlines (after Villaescusa et al, 2003).

7.3 Block model and decline support estimation

The following example has utilised the results of a block model to estimate the ground support requirements for a proposed decline design. A proposed footwall decline design is shown in Figure 12, together with the layout of the proposed stoping areas. The block model has been used to calculate Q values for backs and Qw for sidewalls in all areas of the study area.

The decline design was then intersected with the block model, with the values of Q and Qw being transposed onto the decline. Figure 13 shows contoured values of Q for the backs along the length of the proposed decline design. Using tools within mining software programmes, the total lengths and the start and finish positions of each interval category of Q can then be tabulated.

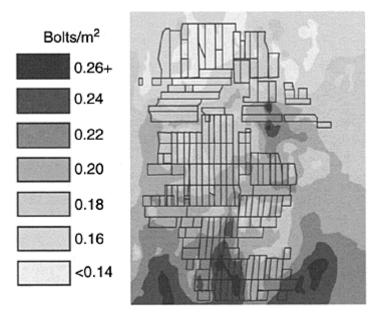


Figure 11. Example of a contoured grid model of the hangingwall surface, showing cable bolt density (bolts/m²), together with stope outlines (after Villaescusa et al, 2003).

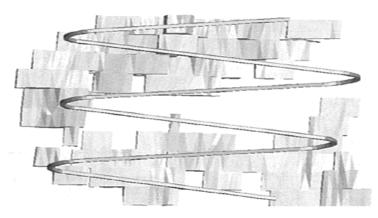


Figure 12. Footwall decline design option showing proposed stoping locations.

Using empirical relationships between Q, span and ESR (excavation support ratio), the ground support requirements for each Q value category can then be estimated (Barton & Grimstad, 1994). This information can rapidly be transformed into an estimated ground support cost for the proposed decline design, as shown in Table 1. By using this technique, the estimated ground support requirements and associated costs for any number of decline designs can be rapidly evaluated.

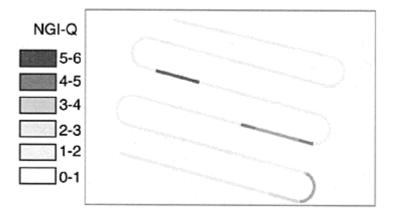


Figure 13. Footwall decline design contoured by Q value for the backs.

8 FURTHER WORK

The use of 3-dimensional geotechnical models and geostatistics in geotechnical engineering is a relatively new and emerging technology. With all new developing technologies, there are initial difficulties and limitations that hopefully should, in time, be overcome.

Some of the current issues relate to the problems associated with developing an integration between the two disciplines of geostatistics and geotechnical engineering. To take full advantage of this integration requires a great deal of skill and experience in both fields. At this stage, there are limited individuals who possess skills and experience in both fields. Some of the examples presented in this paper have been the result of a team approach involving members from both disciplines.

Other issues are intrinsically related to the nature of geotechnical parameters, which are perhaps dissimilar to the variables traditionally modelled in resource discipline. It is considered that more experience is required in the use of geostatistics by geotechnical engineering practitioners before we can develop a comprehensive understanding of what can and can't be done with geostatistics.

Backs* Q range	Length (m)	Weighted average Q	Support category**	Bolt spacing (m)	Cost/m (A\$)	Cost (A\$)
0.0–1.0	0	0.00	_	_	-	_
1.0-2.0	1,867	1.84	4	1.4	655	1,222,138
2.0-3.0	41	2.61	4	1.5	607	24,903
3.0-4.0	33	3.74	3	1.6	320	10,568
4.0–5.0	38	4.24	1	2.0	242	9,191
5.0-6.0	52	5.27	1	2.0	242	12,578
Totals	2,032	2.02				1,279,379
Walls Qw range	Length (m)	Weighted average Q_w	Support category**	Bolt spacing (m)	Cost/m (A\$)	Cost (A\$)
0.0-1.0	0	0.00	-	-	-	_
1.0-2.0	0	0.00	-	-	-	_
2.0-4.0	3	3.44	4	1.6	1141	3,422
4.0–6.0	1,864	4.60	1	2	309	575,510
6.0-8.0	41	6.54	1	2	309	12,659
8.0–10.0	33	9.34	1	2	309	10,189
10.0– 12.0	38	10.59	1	2	309	11,733
12.0– 14.0	52	13.18	1	2.5	225	11,679
Totals	2,032	5.05				625,191
Summary—Backs & Walls Total length (m)		Average support cost/m (A\$)		Total cost	(A\$)	
2,032			937		1,904,569	

Table 1. Example of ground support estimation for proposed footwall decline design option.

* Excavation support ratio=1.6, span=5.5 m.

** After Barton and Grimstad, 1994.

8.1 Geotechnical parameters

There is still some concern amongst geotechnical engineering practitioners whether the fundamental geostatistical assumptions apply to all geotechnical parameters (Sullivan, pers. comm. 1997). For example, traditionally geostatistics seeks to highlight and

estimate *high* positive values of grade in a "background" of no values. Conversely, for some geotechnical parameters, such as RQD, it is more important to highlight and estimate the *low* values in a background of high positive values.

It is assumed that geotechnical parameters are essentially regionalised variables (i.e. spatially correlated), and as such, the use of geostatistical analysis methods should be applicable to these parameters.

Some initial work on spatial correlation of joints has shown that this assumption does hold true for certain geotechnical characteristics (Villaescusa and Brown, 1990). Many authors have already investigated the use of geostatistics to simulate joint fracture patterns (La Pointe, 1980 and Miller, 1979).

It is recommended that, as geostatistics and 3-dimensional modelling techniques are used more frequently in the geotechnical engineering discipline, the case study database is analysed and the most appropriate methods for modelling the various geotechnical parameters investigated. This may involve developing a "suggested method", possibly listing the most appropriate semi-variogram models and interpolation techniques for each geotechnical parameter.

8.2 Orientation bias and anisotropy

Sampling bias is an important aspect to consider in generating 3-dimensional geotechnical models. Certain geotechnical parameters, such as RQD and fracture frequency, can be described as vectors, as their value varies in magnitude depending on the direction it is viewed. The fracture frequency of a rock mass varies in different directions (i.e. anisotropy) and is dependant on the number of joint sets and spacing of each set. This is illustrated in Figure 14, where a fracture

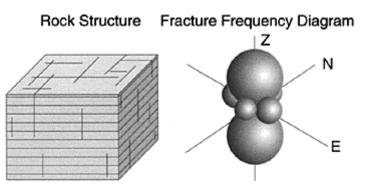


Figure 14. Idealised structured rock mass showing three unequally spaced orthogonal joint sets, together with the corresponding fracture frequency diagram (after Windsor, 1997). frequency diagram (after Windsor, 1997) for a rock mass with three orthogonal, yet unequally spaced, sets is shown.

If the characteristics of the discontinuity or structural regime are not adequately understood, fracture frequency and RQD values chosen can therefore introduce bias within the model. For example, RQD values collected from drill core may be heavily biased, depending on the predominant drilling direction, with respect to the direction of anisotropy. Care must be taken to ensure that subsequent rock mass classifications are not also influenced by this bias, and that appropriate adjustments for sampling bias are made.

This type of bias can be quite difficult to incorporate into a 3-dimensional model, as the direction of mining, or the orientation of a particular excavation surface with respect to local anisotropy is not known until the design of the excavation is decided.

8.3 Rock mass classiflcations

Certain parameters within a number of rock mass classification systems require a certain knowledge about the orientation of the proposed excavation surface. For example, Jr and Ja "…relate to the surface most likely to allow failure to initiate" (Barton et al, 1974). In other words, both the characteristics of a defect and its orientation with respect to the proposed excavation surface must be known. Again, this makes it difficult to determine rock mass classifications prior to the orientation of the excavation surface being known. In this case, the following modelling strategies may need to be adopted:

- It may be necessary to make some assumptions about these parameters (e.g. selecting the worst case Jr/Ja combinations), however, this may result in conservative rock classifications being developed,
- Develop a methodology that can account for these parameters "on the fly" (i.e. obtain information about the proposed excavation design surfaces as they interact with and/or intersect the 3-dimensional model and then calculate rock mass classifications).

8.4 Justification for using 3-dimensional models

Firstly when deciding whether or not to pursue developing a 3-dimensional model, the amount of time and effort in producing a 3-dimensional rock mass model against the perceived benefits gained must be evaluated.

In this regard, further work is required to determine the critical amount of data required that can justify the use of 3-dimensional modelling techniques. To date, the author's experience has shown that, in order to develop a sufficiently robust 3-dimensional block model, basic geotechnical data (e.g. intact rock strength and fracture frequency data) needs to be in the order of *at least* 35%–40% of total geological resource data. This is based on the author's experience in the Australian mining industry.

The importance of developing a 3-dimensional model may decrease as the amount and quality of geotechnical data (i.e. richness) increases as project development progresses. To this end, it may also be important to determine when the benefits of using such models begin to diminish.

8.5 Possible future applications

Some of the inherent properties of geostatistical estimation techniques (i.e. kriging) can allow the engineer to highlight areas in the rock mass where large uncertainties exist in modelled parameters. Once the criticality of geotechnical parameters to the engineering design are established, these estimation techniques may be quite useful in assisting the geotechnical engineer in the optimal planning of geotechnical data collection programmes.

The examples shown have also illustrated that 3-dimensional models can also be used to represent the results of geotechnical design methodologies (albeit empirical). Kriging techniques could also be used to estimate the error, and hence map the *spatial* level of confidence in the results of these analyses.

Another potential use of 3-dimensional rock mass models is in the area of numerical modelling and analytical methods. There is a potential for using the results of 3-dimensional models as direct input (i.e. electronic importing) into numerical modelling and limit equilibrium software. This would allow for spatial variability in the rock mass to be accounted for using these techniques. The use of these spatial models as input data may result in providing smaller, and possibly more realistic, variations in the outcomes.

9 CONCLUSIONS

The use of electronic databases, in conjunction with pre-prepared macros within mining software, has made the development of 3-dimensional geotechnical models an attractive alternative to traditional rock mass characterisation techniques. It has been found to be an efficient method to highlight the variability in rock mass conditions, especially where there is a significant amount of high quality and properly organised data and in relatively complex geological environments.

The modelling methods described above are typically more appropriate in determining ground support requirements in Prefeasibility to Feasibility level studies. Although these models have proved useful in estimating the range of likely ground support requirements, they are not intended to replace detailed analytical ground support design techniques.

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Geotechnical block modelling at BHP Billiton Cannington Mine

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ABSTRACT: A geotechnical block model, based on drillhole geotechnical logs was created for the BHP Billiton Cannington lead-zincsilver mine in North Queensland, Australia. The purpose of the model is to assist the Geotechnical Engineer in medium to long term planning of support costs and mining rates. The first run of the model, as described, was essentially a proof-of-concept trial to test the validity of using numerical interpolation as a means of estimating rock mass conditions. The model output provided insights into the large-scale structure of the Cannington rock mass, and has proved that the block model concept can provide a useful tool in mine planning. The model is currently in the early stages of integration into Cannington's mine planning process.

1 BACKGROUND

1.1 Background of the project

The BHP Billiton Cannington Mine is located in North-western Queensland, approximately 240 km from the town of Mt Isa. The mine location is shown in Figure 1.

Annual production from the mine is approximately 2.4 MT of lead-zinc-silver ore. All of the ore is produced by underground sub-level open stoping. The orebody is a Broken Hill Type massive sulphide deposit. The mineralisation is hosted by high-grade metamorphic rocks in a steeply easterly dipping synclinal fold.

The importance of Geomechanics in the operation of the mine was recognised at an early stage in the mines operations, and led to the integration of geotechnical data collection and assessment throughout the mining process. All development and stope excavations undergo geotechnical assessments at the planning stage prior to being released for construction. During mining, geotechnical assessments are carried out, including geotechnical mapping of development faces, stope monitoring, and recording of abnormal ground conditions. Post-mining assessments are carried out as part of the stope reconciliation process.

An extensive geotechnical database of diamond drillhole logs, face mapping records, and stope stability assessments exists.

The majority of the 12.5 m×12.5 m-spaced resource diamond drilling has been logged geotechnically for: Rock Quality Designation (RQD), Fracture



Figure 1. Location.

Frequency (FF), and Number of Joint Sets (Jn). In all, the drillhole geotechnical database contains records for approximately 220,000 metres of drilling. The drillhole data is used in medium and long term planning estimates of: ground conditions, development mining rates, and support costs. The existing method of estimation involves manual compilation and interpretation of the drillhole data and geology on two dimensional plans and sections. This process produced valid results, but is laborious and time consuming for geotechnical staff. In addition, the results were often not directly transferable to the design engineer as mining parameters.

It was decided to investigate the possibility of modelling the drillhole geotechnical data using some of the common block modelling methods used spatial interpolation of drillhole data. The purpose of the study would be to establish if the modelling techniques were a valid method for estimating rock mass characteristics, and if so, to develop such a model into a mine planning tool.

2 OBJECTIVES AND WORK PROGRAM

2.1 Program objectives

The program objectives were:

- Evaluate the quality of the drillhole geotechnical data for modelling purposes.
- Establish the modelling parameters and constraints.
- Construct a geotechnical model of the rock mass.
- Validate the model output against known conditions.
- Develop the model into a predictive tool for estimating rock mass conditions for medium-to-long term mine planning purposes.

2.2 Work program

The work program comprised:

- Compile relevant geotechnical and geological data.
- Validate the input data.
- Select model constraints, assumptions, input and output fields, and interpolation method.
- Create a geotechnical block model.
- Check the model output against input assumptions.
- Compare the model output to known conditions.
- Develop the model as a planning tool.

Development of the model as a planning tool is an iterative process. It was recognised that the current model would be constructed in a simple fashion in order to avoid introducing any pre-conceived bias by way of untested assumptions. It was recognised that the model would evolve as knowledge was gained from the initial model run.

2.3 Project team

The project team consisted of Mr. Dale Luke, Ms. Tania Kennedy, and Mr. Neil Leggo. Mr. Alan Edwards is currently integrating the geotechnical model with the site resource block model.

3 PROGRAM RESULTS

3.1 Data compilation

The relevant geological and geotechnical data was compiled, including drillhole data, geological wireframes, and historical records of stope stability assessments.

The Cannington deposit is drilled out on a nominal 12.5 m \times 12.5 m pattern of diamond drill holes for reserve estimation.

Diamond drilling is logged geotechnically on a per-metre basis. Logging is generally of a high standard, and logging procedures have been kept consistent from the early stages of the project. In all, the database contains geotechnical data for around 220,000 records. Each record contains a value for:

- Rock Quality Designation (RQD)
- Fracture Frequency (FF)
- Number of Joint Sets (Jn)
- Rock Type

Geological drillhole data includes:

- Ore Grade
- Alteration
- Structure

Geological mapping at 1:250 scale exists for all underground development. This was used as a reference to assist in geotechnical domain boundary selection.

Three dimensional wireframe models of the major rock units have been created by the site geologists. These are regularly updated, and are taken as the best available interpretation of the rock mass geometry.

All stope designs are assessed using the Mathew's Stability Chart Method, (Stewart and Forsyth, 1995). The historical data from these assessments was used as a reference for ground conditions.

The above data was compiled and formed the basic data set for the investigation.

3.2 Drillhole validation checks

A series of simple checks were made, using filter and sort, to ensure that the data was within the range of possible values. A small percentage, <0.01%, of RQD values were outside the 0–100 range. Jn and FF values were found to be within the range of possible values, although no value was recorded for around 25% of the Jn records, corresponding to early drilling where Jn was not recorded.

Raw data was plotted on screen using DATAMINE and compared on a section-bysection, and level-by-level basis with the known geology to see if the expected trends were visible in the data set. Visual checking established that broad geotechnical trends were present, and that there appeared to be good to fair correlation between RQD, Jn, and Fracture Frequency (FF). In particular, the known trends of high and low RQD, relating to the main ore zone and the main fault trends respectively were found to be evident in the raw data. This provided some confidence in the validity of the data set, and the process of interpolation between points.

3.3 Selection of modelling parameters

The Cannington orebody is folded about a steeply easterly dipping isoclinal fold hinge that plunges shallowly to the south. The mineralization is concentrated in metasediments and mafic rocks folded about a central core of amphibolite. The outer part of the fold is barren metasediments and gneiss. The folded sequence is cut by major faults to the south and north.

The deposit has been largely drilled out from underground, in fans spaced 12.5 m along strike. A cross section of the geology showing a typical drill pattern is shown in Figure 2. Geotechnical data has been recorded all drilling, with the exception of a 60 m thick horizontal layer of Cretaceous overburden, and some early exploration holes.

3.4 Model input

The model was to interpolate the values of:

- RQD
- FF
- Jn
- Q
- Q`

RQD and FF are recorded explicitly as numeric data in the database and required no modification. Jn was recorded as alpha data and was converted to numeric data in accordance with the input values for the Q system of Barton et al. (1974) according to the Table 1.

Compatibility with the Stewart and Forsyth (1995) Mathew's Method of Open Stope Design requires input for joint alteration, joint roughness, rock strength, and stress inputs, as well as geometrical inputs which are specific to the stope design. Alphanumeric data for alteration exists in the database; however this data cannot be readily correlated to joint alteration. The relationship between the alteration data and the correct modeling input was judged to be complex and poorly understood, and so provision was made in the model for calculation of the values at a later stage. As an interim measure, default values were set for the respective geotechnical units. Default values were set for Stress Reduction Factor (SRF), Joint Alteration (Ja), and Joint Water (Jw) based on knowledge of local conditions. This was done mainly to make provision for further refinements in the model, rather than a serious attempt to calculate Q or Q`. Notes were made as to the likely geometry of the critical structural features for the review of the Design Geotechnical Engineer.

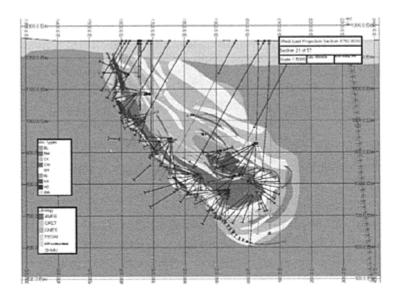


Figure 2. Cross section view of the Cannington orebody, showing typical drill density.

T 11	4	т	•	•
Table	Ι.	Jn	numeric	input.
1 4010		• • • •		1110

Number of joint sets	Code	Jn value	
0	0	0.5	
Few/random	R	1	
1 set	1	2	
1 set+random	1 R	3	
2 sets	2	4	
2 sets+random	2R	6	
3 sets	3	9	
3 sets+random	3R	12	
4 sets	4	15	
4 sets+random	4R	20	
Crushed rock	4 R	20	

3.5 Model constraints

The Cannington orebody has undergone several phases of deformation during its geological history. The resulting strain in the rock is far from uniform. High strain zones

related to geologically early phases of high temperature and pressure ductile deformation are overprinted by later phases of brittle/ductile, and still later phases of brittle deformation. The end result of the deformation is a highly inhomogeneous distribution of fractures. In addition, some rock types because of their strength, melting temperature, age of emplacement, or chemical composition have undergone primarily brittle, joint-forming fracturing, whilst others have undergone ductile deformation leading to the formation of schistose foliation. Late stage brittle faulting has affected all rock units to some degree.

Observation of the fracturing of the rock mass led to the conclusion that the main controls on the nature and degree of fracturing were:

- Rock mineralogy.
- Ductility contrast with adjacent rock types.
- The presence of large-scale regional faults.

Rock mineralogy affects the response of the rock units to deformation resulting from the high temperature/ stress conditions applied during the folding and metamorphism of the early geological history of the deposit. High temperature-of-formation and mechanically strong rocks such as the core amphibolite formed a myriad of brittle structures with strongly altered joint surfaces. Chemically simple rocks such as quartzites appear to have deformed in a more or less ductile fashion, or been overprinted by later silica alteration, forming mineral banding with few joints or foliation planes. Chemically susceptible rocks such as gneiss have undergone intense strain partitioning due to ductility contrast at the contact with stiffer rocks such as the ore and quartzites, and in high strain zones such as the basal fold hinge. High strain in these zones has caused the formation of a strongly developed foliation, joints, and locally, intense alteration of the rock to form sericitic schist bands.

The fracture properties of the major rock units are tabulated in Table 2.

The dominant mechanism listed in Table 2 has been overprinted by late stage brittle fracturing due to faulting as described below.

Regional post-metamorphic deformation has caused faulting across the entire deposit. Two dominant faulting directions occur. These are:

- Major regional faults: Two major faults, referred to as the Trepell and Hamilton Faults cut across the Cannington orebody. These are steeply north-easterly dipping brittle faults. The thickness of the faults varies, as does the degree of fracturing of the rock. The faults are between 20 m and 100 m horizontal thickness, and cut across all rock units. The Trepell Fault can be traced for several kilometres on aeromagnetic images, and is in all probability a deep crustal lineament. The origin of the Hamilton Fault is less certain; however it is likely to be a large splay off the main Trepell Fault. Displacement is left lateral, north side up with movement in excess of 200 m.
- Steeply north-westerly dipping faults. These faults are conjugate to the two major regional faults, and form a cross-linking fracture network between the Trepell and Hamilton Faults. The width and infill in the bird faults varies greatly depending on the properties of the host rock and the amount of preexisting structure. Widths are typically 1 m to 10 m. Infill is generally breccia and/or chlorite schist. Fault displacements are right lateral, west side up and can be of the order of tens of metres.

Rock type	Mechanism	Intensity	Joint alteration
Amphibolite	Brittle	High	High
Ore	Ductile	Low	Low
Quartzite	Brittle/Ductile	Mod	Low
HW Gneiss	Brittle/Ductile	High	High
FW Gneiss	Brittle/Ductile	Mod-High	Mod

Table 2. Rock fracture properties—pre-faulting.

The geometry of the major rock units and structures is known, and has been modeled in considerable detail by the site geologists. Given that rock type, and the presence of late-stage brittle faults were known to be the primary controls on the fracture geometry and density the model space was divided into broad geotechnical domains according to structure and rock type. Five structural domains, each with composite rock-type sub-domains were defined. These domains were modeled using a simplified set of wireframes taken from the resource-modeling boundary wireframes created by the site geologists. These wireframes were accepted as being the best available models for the rock-type and structural geometries. It was necessary to simplify the model by excluding wireframes for minor rock types, or those with similar mechanical and fracture properties to the surrounding host rock. This was done to avoid spurious interpolations in narrow zones where there was little data. It was recognised that this would have the effect of reducing the model resolution in some areas, but this was judged to be preferable to placing distortions in the broad-scale model interpolation as a result of the constraining effect of narrow zones that were of only local significance.

Each domain was taken as being a separate entity. In practice this assumption is seen as reasonable given that the adjacent units are known to have differing mechanical and fracture properties which are non-gradational across the contacts. The model interpolation is constrained by the structural and geological wireframes, with no interpolation across the domain boundaries.

The model was divided into five structural domains based on the location of the Trepell and Hamilton Faults. Within each structural domain, there are rock type sub-domains. In all 5 structural domains, containing a total of 13 sub-domains was defined as shown in Table 3.

The location of the domains is shown in plan view in Figure 3

Drillhole density varies from domain to domain to domain. The most uniform data distribution is centered on the mineralization. The majority if the drilling has been carried in fans from underground drives. This means that some sections of the rock mass have

Structural domain	Lithological sub-domain	Code	Description
North Zone	Nil	11–14	Undifferentiated mine sequence
Trepell Fault	Nil	21–25	Undifferentiated fault zone

Table 3. Model domains.

Central Zone			
	HW Gneiss	31	Altered gneiss
	FW Gneiss	32	Altered gneiss
	Amphibolite	33	Amphibolite
	Quartzite	34	Barren quartzite
	Ore	35	Mineralisation
Hamilton Fault			Highly fractured
			Mine sequence
	HW Gneiss	41	Altered gneiss
	FW Gneiss	42	Altered gneiss
	Amphibolite	43	Amphibolite
	Quartzite	44	Barren quartzite
	Ore	45	Mineralisation
Southern Gneiss	Nil	51–55	Undifferentiated gneiss

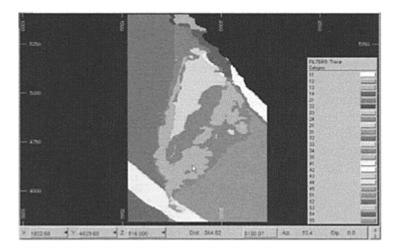


Figure 3. Geotechnical domains, plan view, 350 m level.

data "clumps" around the drill collar points, while other sections of the rock mass are relatively uniformly sampled, albeit at varying density depending on their distance from the drill hole collar points. Because of the model constrains and variable data density, care is required when interpreting the results to ensure that the effect of data density is considered when assessing the model output.

3.6 Interpolation

The results of the model variography were inconclusive, with no clear global preferred orientation. Knowledge of the structural geology of the deposit indicated that directional interpolation between points would vary in different structural domains, and even within different rock types within the structural domains. After much deliberation, it was decided that neither the best orientation for a search ellipse, nor the best mathematical approximation to the spatial variation in the fracture distributions were known. For this reason it was decided to use a simple interpolation for the first run, and see what results were produced. Isotropic Inverse Distance Weighted (IDW) with an index value of 2 was selected as the interpolation algorithm for the first-pass.

Parent cell sizes is (X, Y, Z) 64 m, 50 m, 64 m. Daughter cell size is 8 m, 12.5 m, 8 m. The model was run in three passes with search radii of 30 m, 60 m and 150 m respectively.

The model was constructed and run using DATAMINE software.

4 MODEL RESULTS

Model output values were visually compared to the drillhole values by simultaneously loading the model and drillhole values. Broad trends in the data were apparent in the interpolated cell values, and these adequately reflected the broad structural and geological trends. It was found that the interpolated values for cells containing drillhole data points reflected the drillhole data. The variability of the interpolated values between drillholes is less than that of the source data, indicating that the modelling has smoothed the output. This has the effect of reducing the reliability of the model on a local scale. In general, the model cell values of RQD and Jn can be taken as a reasonable approximation to the actual rock mass on a scale of tens to hundreds of metres. Care is required when using the interpolated values on a drive scale. Reference should be made to the source data with respect to data clustering and drill hole proximity when applying the model results to local areas.

The calculation of Q and Q` was carried out as the product of the cell values for RQD, FF, Jn, Jr, Ja, Jw and SRF, such that:

Q`=RQD * 1/Jn *Jr *1/Ja Q=RQD * 1/Jn * Jr * 1/Ja *Jw *1/SRF.

The values that were generated were heavily influenced by the default values input for SRF, Jw, Ja, and Jr, and are not considered to be in any way indicative of the true values for Q and Q° , and are not considered further in this discussion. Later models may incorporate refinements that could improve the modelling of these parameters.

The model domains were used to extract and analyse the source data for the input records according to lithological sub-domain. The results showed that the sub-domaining had effectively separated out rock types with distinct data distributions. The results confirmed the assumption that the units would have differing fracture characteristics related to the mechanical and chemical properties of the rock.

The units were classified according to gross chemical composition as mafic, quartzitic, or feldspathic. A fourth group of "altered" composition relates to the component of the

rock mass which has been substantially altered either chemically or physically from its original state. This group is normally the component of the sub-domain displaying the worst ground conditions.

Graphs of the domain distributions for rock group, RQD, FF, and Jn are presented in Appendix I.

The model output was compared with the actual ground conditions in the mine. The correlation between the model values and the actual conditions was good on a large scale (100's of metres), particularly in defining broad fracture zones. Correlation on a scale of 10's of metres was fair to good, depending on data density and the variability of the rock mass. Interpolated values were not considered to be reliable on a scale of less than 25 m.

All development headings are mapped for Q value during mining. The resulting Q values are used to categorise the rock mass conditions as either "Good", "Poor", or "Very Poor" based on the divisions shown in Table 4. The assessment is used as a guide to ground support selection, based on three standard support categories.

The support requirements are currently under review and are not detailed here. However, the level of ground support (and cost) increases from Support System A through to Support System C as the level of support increases in response to poorer ground conditions. The model output was compared to the mapped Q values to assess the ability of the model to correctly predict the support costs for a given area.

Figure 4 shows the comparison between the model Q output and the Q values recorded by mapping of the development face during mining of two separate headings located 475 m below surface.

- Heading 46XC is located within the Hamilton Fault Quartzite Domain (44). The rock mass is characterised by highly fractured quartzite, moderate to strong joint alteration, low to moderate water inflows, and moderate stress to strength ratio.
- Heading 54XC is located in the Central Zone Ore Domain (35). The rock mass is characterised by weakly fractured mineralised quartzite, weak joint alteration, dry rock conditions, and moderate stress to strength ratio.

Because the model inputs for Ja, Jr, Jw and SRF are not explicitly contained in the database constants were applied to approximate the Q value in the model. The default values are the "best estimate of the average" for each parameter. As a result they serve only to place the model output in the correct approximate range. Within each domain, all variation in the model Q values is as a result of variation in RQD and Jn values. The default values are shown in Table 5. The purpose of this example is to illustrate the general

Q range	Description	Bolting	Surface support
Q>4.0	Good	Pattern A	Type A
1.0 <q<4.0< th=""><td>Poor</td><td>Pattern B</td><td>Type B</td></q<4.0<>	Poor	Pattern B	Type B
Q<1.0	Very Poor	Pattern C	Type C

Table 4. Q value and ground support specification.

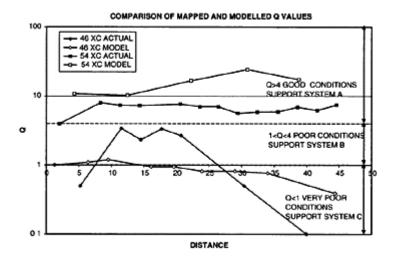


Figure 4. Comparison of modelled and actual Q values.

Table 5. Model default values.

Domain	Code	Jn		Ja	Jr	Jw	SRF
Central Ore	35		6	2	2	1	0.7
Hamilton Fault	44	1	12	6	2	0.7	0.7
Quartzite							

correlation between the model and values obtained from mapping. The heavy reliance on estimated global constants in the calculation of model Q highlights the desirability of collecting defect data during drill core logging. Such data would allow far better estimation of the true distribution of Q values.

The modelled values show evidence of smoothing when compared to the actual values from the mapping. The face mapping has an effective resolution of 3 m, this being the nominal distance between data points, and hence is better suited to detecting local changes in rock mass conditions. The model output provides a reasonable approximation to the actual "average" ground conditions and support requirement for the two drives, and is adequate for forward planning estimates of cost and advance rate. Improved estimation of the input values for Jn, Ja, Jr, Jw, and SRF is required to improve the correlation with actual conditions. Ideally these values will be derived from mapping records, or from drill logs of alteration and joint defect characteristics and incorporated into the model as variables rather than as global constants for each sub-domain.

5 FURTHER WORK

At the time of writing the geotechnical model was being incorporated into the site resource model, as part of a process of integration into the overall mine planning process.

In its current state, the model can be used with caution as a predictive tool for mine planning. The model can save many hours of time for the site geotechnical staff, provided that the limitations of the interpolated values are understood.

The challenge is to further refine the modelling process to reduce the smoothing effects, incorporate better estimates of Q value inputs, and improve the directional interpolation. The modelling output will improve with the application more sophisticated modelling techniques, improved knowledge of the distribution of various Q value inputs, and further back analysis of the output.

In the longer term, a major improvement in the modelling of Q will come about with the development of alteration, hydrogeological and stress models. These will provide direct inputs into the modelling in the same way that RQD, and Jn do in the current model.

This will allow Cannington to gain the maximum value from the high quality data that exists in the Cannington drillhole database.

6 CONCLUSIONS

The Cannington mine has acquired a high quality geotechnical data by virtue of the efforts of many people from very early in the mine's history. Numerical modelling of the Cannington rock mass has showed that a simple interpolation algorithm, applied to high quality data set, can produce an output that is predictive of rock mass conditions on a large scale. Further development of the model, in terms of refinements in the modelling methods will yield improvements in the predictive ability of the model, and to further assist in the mine planning process.

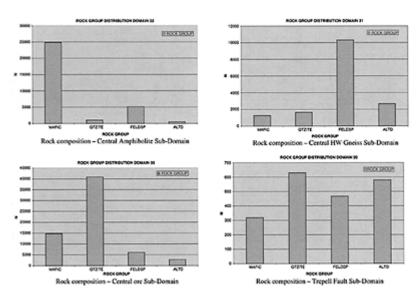
ACKNOWLEDGEMENTS

Permission to publish this paper has been granted by BHPBilliton Cannington.

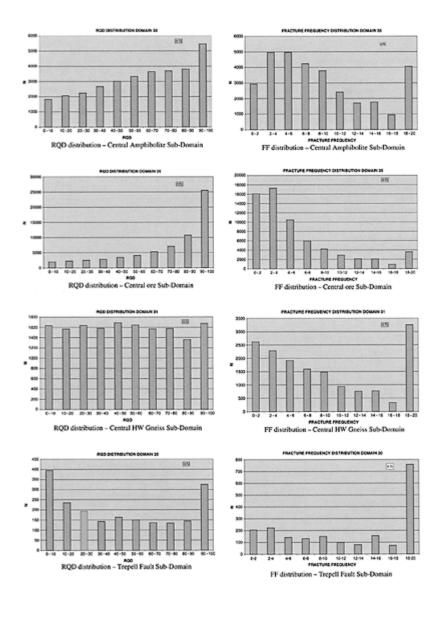
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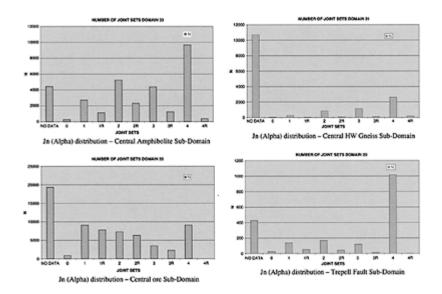
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APPENDIX I





The application of a rock mass rating system at Tau Lekoa Mine

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ABSTRACT: Rock mass rating systems have not been widely applied on South African gold mines, as there is a perception that they do not adequately cater for stress effects. Stress and stress induced fracturing are the dominant factors influencing rock mass conditions in most South African gold mines. Following visits to several mines in the Bushveld Igneous Complex, the authors were of the opinion that rock mass ratings could be successfully applied on Tau Lekoa Mine if stress fracturing could be incorporated. The successful application of a rock mass rating system at Tau Lekoa would assist in determining geotechnical areas. In addition, subjectivity in assessing ground conditions would be reduced and a greater level of consistency achieved. Various rock mass rating systems were evaluated in terms of their applicability to Tau Lekoa conditions and a variation of the O-system was chosen as the most appropriate. Due to the variations in stress conditions and the impracticality of modelling every tunnel and stoping panel, conceptual modelling of various situations was conducted to determine stress look-up tables for various commonly encountered layouts. Currently the emphasis is on collecting sufficient data for the delineation of different geotechnical areas. Back analysis of ground conditions and support systems continues, with the aim of developing definite relationships between support requirements and the rock mass ratings.

1 INTRODUCTION

1.1 Location and description

Tau Lekoa Gold Mine (AngloGold Ashanti Limited) is situated approximately 170 km southwest of Johannesburg, near Orkney in the North West Province of South Africa. The mine has been in production since 1991. A scattered mining strategy is employed on

the mine due to the geological complexity of the orebody Stoping operations take place on strike and mining is conducted between 900 m and 1650 m below surface.

1.2 Geology and geotechnical environment

The Ventersdorp Contact Reef (VCR) is mined at Tau Lekoa Mine. The VCR lies unconformably on the Gold Estate Formation of the Central Rand Group and is overlain by the Klipriviersberg Group of the Ventersdorp Supergroup. The orebody can be described as tabular, dipping at $\pm 30^{\circ}$ towards the northwest. The channel width varies between 10 cm and 300 cm and reef rolls are common.

Several major faults striking mainly northeast to southwest and dipping southeast disturb the area. Quartz and calcite veins, dykes and joint sets also intersect the reef. The veins are often flat dipping and are especially hazardous. Two major joint sets have been identified, one striking north to south and the other northeast to southwest. Both sets are steep dipping at 70–90 degrees. Low angle thrust faulting has also been observed. In some areas a mylonite filling is present between the lava and the VCR.

1.3 Rock mass ratings in an intermediate and deep mining environments

Rock mass rating systems have not been widely applied on South African gold mines. The reasons for this are unclear, although there is a perception that rock mass rating systems do not adequately cater for stress effects. Stress and stress induced fracturing are the dominant factors in rock mass conditions in most South African gold mines. Piper (1985) evaluated five different systems in terms of their application in deep mines. He concluded that whilst rock mass classification systems can provide valuable information in the design and support of mining excavations, the systems evaluated had disadvantages, which limited their application in deep mines.

Following visits to several mines in the Bushveld Igneous Complex (BIC), the authors were of the opinion that rock mass ratings could be applied on Tau Lekoa Mine if stress fracturing could be incorporated. The application of a rock mass rating system would provide a consistent approach to classifying ground conditions and give a methodology for support design.

2 EVALUATION OF ROCK MASS RATING SYSTEMS AT TAU LEKOA MINE

Several rock mass rating systems were initially reviewed including the following:

- Geomechanics Classification (RMR) system (Bieniawksi, 1973)
- Modified Geomechanics Classification (MRMR) system (Laubscher & Taylor, 1976)
- Q-system (Barton et al., 1974; Grimstad & Barton, 1993).

Watson & Noble (1997) indicated that a modified Q-system provided the most accurate description of observed conditions. York et al. (1998) came to a similar conclusion in that the Impala Mine modified Q-system provided the most accurate description of actual

observed conditions. Both of these systems were similar in that the stress reduction factor (SRF) had been modified.

Piper (1985) had indicated that whilst rock mass classification systems had been developed to cater for jointed rock masses further development was required for their application to fractured rock masses as encountered in deep and intermediate mines. Kirsten (1988) had proposed modifications to the Q-system to account for fracturing of the rock mass. Dukes & Laas (2002) applied this modified system in several tunnels in a deep mining environment.

It was decided to apply the different Q-system variations to a number of different situations to determine the most appropriate for Tau Lekoa conditions. The following systems were applied:

- Impala Mine Q-system (Human, 1997)
- Modified Kirsten Q-system (Kirsten, 1988)
- Q-system (Grimstad & Barton, 1993)
- Tau Lekoa Q-system (Dunn & Hungwe, 2002).

The initial work by Dunn & Hungwe (2002) indicated the Q-system (Grimstad & Barton, 1993) and the Modified Kirsten Q-system (Kirsten, 1988) seemed the most appropriate and these were used for a more detailed study. Minor modif ications were made to the descriptions used in the Q-system to make it easier to apply and more meaningful in the Tau Lekoa geotechnical environment.

2.1 Overview of the Q-system

The Q-system is regarded as one of the most comprehensive rock mass rating systems and is widely used. A description of the system follows and the differences between the Q-system (Grimstad & Barton, 1993) and the Modified Kirsten Q-system (Kirsten, 1988) are outlined.

The Q value is determined from the following relationship (Barton et al., 1974): $Q=(RQD/J_n)\times(J_r/J_a)\times(J_w/SRF)$

(1)

where RQD=rock quality designation; J_n =joint set number; J_r =joint roughness; J_a =joint alteration; J_w =joint water; and SRF=stress reduction factor.

This can be expanded to the following definitions:

• RQD/J_n represents the block size

• J_r/J_a represents the minimum inter-block shear strength

• J_w /SRF represents the active stress.

In an intermediate to deep mining environment, stress induced fracturing is more prevalent than in a shallow mining environment and hence plays a major role in the rock mass stability. As stress induced fractures contribute to block size they are considered as joints and accordingly incorporated as a joint set in the joint set rating.

2.1.1 Rock quality designation (RQD)

RQD (Deere, 1969) attempts to quantify discontinuity (joints and stress fractures) density. Table 1 shows the guidelines used to determine RQD. Usually RQD is determined from core but it is possible to estimate RQD from the joints or fractures per unit volume from rock exposures. Joints and fractures are counted to determine the fracture frequency per cubic metre (FF/m³) or joints per volume (J_v) when a rock exposure is available (Palmström, 1982). The following relationship is used to approximate RQD to joints per volume (J_v) determined from underground examination of rock exposures.

RQD=115-3.3 J_v

(2)

where J_v =joints per volume (m³).

RQD		FF/m ³	Rating
Very poor	Highly sheared, brecciated, fractured rock	>28	0–25
Poor	Highly veined, bedded, weak layer, altered	21-27	26–50
Fair	Medium veined or bedded	13–20	51-75
Good	Competent rock with a few discontinuities	8-12	76–90
Excellent	Competent rock with no/very few discontinuities	<7	91–100

Table 1. RQD descriptions for Tau Lekoa.

2.1.2 Joint set number (J_n)

This is a measure of the number of joint sets observed in the area mapped. The influence of random jointing is also incorporated as shown in Table 2.

2.1.3 Joint roughness number (J_r)

Joint roughness describes the frictional characteristics of the joint walls in as far as they oppose relative movement along interacting rock surfaces. A rating for rough, smooth and slickensided joint walls is indicated in Table 3.

2.1.4 Joint alteration number (J_a)

Joint alteration refers to any filling along discontinuity planes. The thickness and strength of the filling determines the discontinuity strength of the joint and its ability to resist sliding. Table 4 summarises guidelines used to rate the various categories of infilling and their characteristics.

2.1.5 Joint water (J_w)

The presence of water reduces the friction or weathers the filling in joints, hence decreasing the interblock stability. Ratings for dry and water-dripping joints are shown in Table 5.

Joint set number (J _n)	Rating
No joints	1
One joint set	2
One joint set & random	3
Two joint sets	4
Two joint sets & random	6
Three joint sets	9
Three joint sets & random	12
Four joint sets or more	15
Sheared rock	20

Tal	ble	2.	Joint	set	num	ber	(\mathbf{J}_n)).
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Table 3. Joint roughness (J_r).

Joint roughness (J _r)	Rating
Stepped	
Rough	4
Smooth	3
Slickensided	2
Undulating	
Rough	3
Smooth	2
Slickensided	1.5
Planar	
Rough	1.5
Smooth	1
Slickensided	0.5

2.1.6 Stress reduction factor (SRF)

SRF attempts to quantify and incorporate the influence of zones of geological weakness, stress or both. In an environment where geology is the major factor influencing stability the original Q-system (Barton et al., 1974) or the Impala Q-system (Human, 1997) are adequate.

For competent rock with stress problems, Barton et al. (1974) related the SRF to the ratio between the uniaxial compressive strength (σ_c) of an intact rock and the major principal field stress (σ_1) as shown in Table 6. The range of SRF values was found to be inadequate and an expanded range was introduced by Grimstad & Barton (1993) as shown in Table 7.

Due to the fact that both geology and stress influence stability, an attempt was made to combine the

Joint alteration (J _a)	Rating
Tightly healed	0.75
Joint walls stained only	1
Thin filling (<5mm)	4
Thick filling (>5mm)	6
Shear zone/weak filling	8

Table 4. Joint alteration (J_a).

Table 5. Joint water (J_w) .

Joint water (J _w)	Rating	
Dry	1	
Dripping water	0.5	

Table 6. SRF Barton et al. (1974).

σ_c/σ_1		SRF
0–2.5	Heavy rockburst (massive rock)	20–10
2.5-5	Mild rockburst (massive rock)	10–5
5-10	High stress, very tight structure	2-0.5
10-100	Medium stress	1

$\sigma_c\!/\sigma_1$		SRF
<2	Heavy rockburst (massive rock)	200–400
2–3	Slabbing and rockburst (massive rock)	50-200
3–5	Moderate slabbing after 1 hr in massive rock	5-50
5-10	High stress/very tight structure	0.5–2
10-200	Medium stress condition	1
>200	Low stress, near surface, open joints	2–5

Table 7. SRF (Grimstad & Barton, 1993).

geological and stress influences, however this was not very successful (Dunn & Hungwe, 2002).

Kirsten (1988) postulated that SRF is not a geological property that varies for different regimes of rock around an excavation. It is a parameter that characterises the loosening of the rock around the excavation as a whole. Thus SRF is dependent on the variation of the maximum and minimum (σ_1 and σ_3) stresses relative to rock alteration and the uniaxial compressive strength (σ_c) of the host rock.

Kirsten (1988) argued that the Barton et al. (1974) criteria for determining SRF are open to interpretation, because of the qualitative nature of the criteria and the difficulties that often arise in deciding whether the rock mass is homogenous or not. These problems can be overcome by observing that, in the case of non-homogenous rock, SRF is related to the overall quality of the rock (Q) and in the case of homogenous rock, to the field stress state relative to the rock mass strength.

According to Kirsten (1988), the loosening of non-homogenous rock is determined by gouge on the joints or the infill in the shear zones. For homogenous rock the properties of the blocks demarcated by tightly healed joints determine loosening.

Relationships for homogenous and non-homogenous rock masses are denoted by SRF_h and SRF_n (Kirsten, 1988).

$$SRF_{h}=0.244K^{0.35}(H/\sigma_{c})^{1.32}+0.18(\sigma_{c})^{1.41}$$
(3)
$$SRF_{n}=1.81Q^{-0.33}$$
(4)

where K=maximum to minimum principal field (absolute) stress ratio; H=head of rock corresponding to maximum field stress (m); σ_c =uniaxial compressive strength of the rock.

 Q_n and Q_h are the rock mass quality indices for non-homogenous and homogenous rock masses respectively. They are given by the following relationships (Kirsten, 1988):

$$Q_{n} = ((RQD/J_{n}) \times (J_{r}/J_{a}) \times (J_{w}/1.81))^{1.49}$$
(5)

$$Q_h = (RQD/J_n) \times (J_r/J_a) \times (J_w/SRF_h)$$

(6)

The larger value for SRF from the two determinations and the corresponding smaller value for Q represents the ruling rock mass quality. The two terms represent respectively the stress and structurally controlled behaviour of competent rock.

2.1.7 Rock mass class

The following scale was used to determine the rock quality class:

Very Poor	<1
Poor	1–4
Fair	4–10
Good	10–40
Very Good	>40

3 CASE STUDIES

In order to evaluate the two Q-system versions they were applied to two case studies.

3.1 Case study 1—Crosscut stability

Tau Lekoa crosscuts accessing the tabular reef, often end at a reef intersection. This results in the last 30 m of the crosscut being relatively close to the reef and prone to damage due to stress induced by mining and the relatively low strength of the footwall quartzites of 150 MPa.

Early stoping of the critical area and installation of appropriate secondary support in the crosscut cater for these conditions. However, practical and production constraints sometimes result in this work not taking place. This results in a pillar being left above the crosscut resulting in increased stress levels and damage to the crosscut if the critical level of stress to strength is exceeded.

An analysis of nine different crosscuts was conducted using four different variations of the Q-system (Dunn & Hungwe, 2002). From this initial study it was concluded that the Q-system (Grimstad & Barton, 1993) and the Modified Kirsten Q-system (Kirsten, 1988) provided results that best reflected the actual ground conditions. The determination of SRF using the Kirsten (1988) method appears to be the more reliable and comprehensive in describing what actually occurs to the rock mass during mining operations. Based on these observations it was decided to use the Modified Kirsten Q-system for the remainder of the study.

A total of nineteen crosscuts on several levels were assessed using the Modified Kirsten Q-system. Assessments were conducted at different points in the last 30 m of each crosscut where the distance to the reef varied between 5 m and 25 m. The aim was to determine differences in ground conditions for areas that been over-stoped early compared to those which had been over-stoped late. Installed support was also taken into account.

3.1.1 Numerical modelling

In order to determine the SRF it was necessary to estimate the stress levels acting on each crosscut. As it was envisaged that this type of assessment would become routine, modelling of each tunnel would be too time consuming and onerous. Two conceptual models that reflected the early and late over-stoped situations were constructed and run for different depths. This was used to develop stress look-up tables that could be routinely applied to estimate the stress levels.

Minsim-W, a 3D-boundary element programme was used. This programme assumes the rock mass is elastic, continuous and homogenous and is thus only used for comparative purposes and as an indication of stress levels. The input parameters used are listed in Table 8.

The virgin stress components are based on in situ stress measurements determined on 1200 level (Lombard, 1989). These measurements indicated a relatively high horizontal stress in the north-south direction and a low horizontal stress in the east-west direction.

Table 9 is an example of a stress look-up table for a situation where a pillar is initially created over the crosscut then over-stoped at a later stage. When conducting a Q rating the stresses can be estimated by choosing the situation most similar to the actual situation.

Element width	10 m
Young's Modulus	70GPa
Poisson's ratio	0.2
Stoping width	1.8 m
Reef Dip	25°
Friction angle	30°
Cohesion	0
Virgin stress components	North-south=0.024 MPa/m
	East-west=0.0045 MPa/m
	Vertical=0.03 MPa/m

Table 8. Modelling input parameters.

Table 9. Major principal stress (σ_1 in MPa) estimates at points of interest for different levels when over-stoping is conducted last.

Level	Reef—X/C distance (m)	Abutment on 1 side	Pillar above X/C	X/C Overstoped
900	5	42	64	20
	15	46	81	27

	25	48	73	27
1050	5	49	75	24
	15	53	100	31
	25	56	85	35
1200	5	56	85	27
	15	61	120	36
	25	64	97	40
1350	5	63	96	30
	15		68 140	40
	25	72	109	45
1500	5	70	106	33
	15	76	144	45
	25	80	121	49
1650	5	77	107	37
	15	84	163	53
	25	88	133	54

3.1.2 Results

Most of the crosscuts were not over-stoped first as required, and very poor to poor ground conditions were encountered. Crosscuts that had been over-stoped first, had higher Q values, with some approaching the boundary between poor and fair conditions. Generally, the value of Q decreased with depth indicating deterioration in ground conditions. This is as a result of increased stress and fracturing. Abutments and pillars over crosscuts also had the effect of reducing Q for the same reasons. Unfavourable ground conditions were observed on the upper levels as a result of poor mining sequence.

Poor ground conditions were also observed in tunnels that had been over-stoped when required. This is as a result of damage to the rock mass during overstoping followed by relaxation after over-stoping (de-stressing). These effects could possible be limited by ensuring that adequate secondary support is installed prior to over-stoping. Only four of the nineteen tunnels investigated had secondary support installed. This support was installed after damage had already taken place.

3.1.3 Support design

In addition to indicating the rock mass quality, support requirements can be determined using Q and the equivalent dimension (De) of the tunnel (Barton, 1976). Kirsten (1998) determined the permanent shotcrete thickness (t) and the bolt/anchor spacing (b) from the

following expressions derived from recommendations on support given by Barton et al. (1974).

(7)
$$t=11.25 \times D_e/Q^{0.43}$$

De=S/ESR

$$h=1.21 \times O^{0.17}$$
(8)

$$p=1.21 \times Q^{0.17}$$
(9)

where S=span of the excavation; ESR=excavation support ratio (criticality of excavation and duration of use).

A 3.2 m excavation span is assumed, with an ESR of 1.6, which is generally applicable to mine tunnels. Tables 10 and 11 show theoretical support requirements for the nine tunnels used for the initial study.

Table 10. Support requirements for crosscuts overstoped first using the modified Kirsten Q-system (Dunn & Hungwe, 2002).

Working place	SRF	Q	De	t (mm)	b (m)
1050 S4 x/c 10	5.9	1.7	2	17.9	1.3
1350 S1 x/c9	8.4	1.9	2	17.1	1.3
1500 S1 x/c 10	9.4	1	2	20.8	1.2

Table 11. Support requirements for crosscuts overstoped last using the modified Kirsten Q-system (Dunn & Hungwe, 2002).

Working place	SRF	Q	De	t (mm)	b (m)
1050 S4 x/c 11	19.8	0.5	2	30.3	1.1
1200 S4 x/c 13 13	17.1	0.5	2	30.3	1.1
1200 S4 x/c15	8.6	1.6	2	18.4	1.3
1350 S1 x/c 7	10	1.0	2	22.5	1.2
1500 S3 x/c 17	21.6	0.7	2	26.2	1.1
1500 S1 x/c 8	21.7	0.4	2	33.4	1.0

From Table 10 it can be seen that crosscuts that are over-stoped first require support with ± 20 mm of shotcrete with tendons spaced 1.3 m apart. Table 11 indicates ± 30 mm of shotcrete with tendons spaced 1.1m apart for crosscuts that are over-stoped last.

3.2 Case study 2—Q-system applied to stoping

Following the application of the Modified Kirsten Q-system (Kirsten, 1988) in tunnels it was evaluated in a stoping environment. The Q-system (Grimstad & Barton, 1993) was included in the study. Fifteen stoping panels were evaluated. Q ratings of the lava hangingwall were determined at the top, middle and bottom of each panel face, with the average face length being 20 m. The panels rated covered a wide range of configurations and included both south and north mining panels. South mining panels are regarded as more problematic in terms of ground conditions at Tau Lekoa Mine.

3.2.1 Numerical modelling

As with the tunnel study, it was necessary to estimate stress levels. As stoping is generally more complex it was not practical to model all mining panels. Conceptual numerical modelling was conducted to determine major (σ_1) and minor (σ_3) principal stresses for five different mining layouts (Hungwe & Dunn, 2003).

Minsim-W was again the package of choice and the input parameters were the same as used in the crosscut case study. Stress look-up tables were developed to assist and facilitate easier rock mass ratings.

3.2.2 Results

The two Q-systems rated the same working places into two predominant rock mass classes. Approximately 63% of the working places fell in the Poor classification range with the Modified Kirsten Q-system whilst 44% of the same working places fell in the Very Poor range according to the Q-system. Generally, higher stresses as a result of depth and higher percentage extraction will result in worse conditions for a rock of the same strength with similar natural discontinuities.

Working place	SRF*	Q*	SRF**	Q**
North mining				
1500 S1 Rse 9E P27	8.1	1.4	21	0.5
1650 S3 Rse 19 P19	9.3	1.7	14	1.1
1050 S1 Rse 17 P1	9	2.4	7	3.0
1650 S3 Rse 19 P17	15.1	0.5	95	0.1
1500 S2 Rse 13 P35	8.1	2.7	21	1.1
1500 S2 Rse 13 P37	9.2	2.2	10	2.1
1350 S1 Rse 21 A P13	9.7	2.3	170	0.1
1500 S3 Rse 20 P31	9.2	1.7	12	1.3
1200 S4 Rse 15 P25	27.7	0.6	220	0.1

Table 12. Q rating comparison for North and South mining panels (Hungwe & Dunn, 2003).

Average	11.7 1.7	63.3 1.0
South mining		
1500 S1 Rse 10 P40	10.2 1.3	95 1.3
1500 S1 Rse 14 P24	57 0.2	270 0.0
1500 S1 Rse 10 P30	62 0.3	270 0.1
1500 S1 Rse 11 P16	15.6 0.4	155 0.0
1650 S3 Rse 23 P12	16 1.4	160 0.1
1650 S2 Rse 21 P22	24 0.6	220 0.1
1500 S1 Rse 10 P34	11 2.1	210 0.1
Average	28 0.9	197 0.24

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* Modified Kirsten Q-system (Kirsten, 1988).

** Barton Q-system (Grimstad & Barton, 1993).

There appeared to be very little difference between the two rating systems and this is possible because the range of rock mass conditions was quite limited. The Modified Kirsten Q-system was favoured as it allows you to calculate the SRF, whilst the Q-system is more qualitative and can be open to interpretation and requires a degree of interpolation.

Table 12 summarises and compares the general ground conditions in north and south mining panels visited and rated using the Modified Kirsten Q-system.

An ideal comparison would have been to compare north and south panels with the same mining sequences, spans and at the same depths. However, the comparison indicates that on average, south mining panels encounter poorer ground conditions than north mining panels.

4 DISCUSSION

By embarking on a rock mass classification approach it was possible to compare ground conditions in a consistent manner. The collection of rock mass rating data and representing it graphically on mine plans will assist in identifying different geotechnical areas as well as indicating areas where ground control problems are anticipated.

For the tunnel case study an attempt was made to estimate support requirements. Further work is required in this regard for both tunnelling and stoping. This requires an extensive back analysis of support applied versus the rock mass ratings and documenting successes or failures of the support systems.

The compilation of look-up tables to estimate stress levels makes the process of Q ratings less onerous. However, in cases where the situation differs markedly from the standard, scenarios specific modelling would be required.

Both the Q-system and the Modified Kirsten Q-system resulted in reasonable results that corresponded with actual observations. The Modified Kirsten Q-system was preferred as less judgement is required in the determination of SRF.

5 CONCLUSIONS

The application of the Q-system and the Modified Kirsten Q-system, in an environment where fracturing is predominant, was reasonably successful. Further work is required to refine this approach especially in terms of designing support.

The main advantage of applying a rock mass rating system is a reasonably objective and consistent approach to quantifying ground conditions.

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Determination of rock mass behaviour as an integral part in rock mass characterisation using probabilistic methods

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ABSTRACT: When the geotechnical engineer is involved in rock mass characterisation, he is faced with geological and geotechnical data which inherently contain uncertainties. This paper proposes a procedure to facilitate the processing of these data, and to consistently consider uncertain parameters in the geotechnical design process. Consequences on the requirements for geological investigations and modelling are discussed. The general procedure of rock mass characterisation and its computational implementation is described whereby the focus is set on probabilistic processing of geological data. The application of the procedure is presented in detail on a rock mass characterisation performed for a major tunnel project in Austria.

1 INTRODUCTION

The design of an underground structure always requires a characterisation of the ground within the project area. This characterisation forms the basis for the design of the excavation and support. Characterising a rock mass requires a complete image of the rock material, its condition, geometrical assembly, and its discontinuity and fault systems. Different investigation methods are applied in various investigation phases. These methods can include field mapping, drill hole logging, and geophysical investigations. An investigation campaign should result in a threedimensional geological model representing the most probable rock mass condition. However, due to the limited extent of site investigations, this model contains uncertainties. Field mapping is restricted to the surface, whereas drill cores provide only data for a limited rock mass volume. In addition to the uncertainties of the geological model, the values of the geotechnical parameters of

the rock mass material are often widely distributed. Finding representative values is a challenging task.

The purpose of this paper is to highlight the aims of the geological investigation, the associated geological modelling and data evaluation, and to propose a characterisation method which allows the consideration of the inherent uncertainties of the geological model and geotechnical parameters. A procedure for rock mass characterisation is presented which was released as a guideline in Austria. This procedure forms the framework for further analyses. The steps of the procedure are practically formulated in a computational model which contains the relevant information of the geological model and data evaluations. Probabilistic methods were used to evaluate the computational model. The general principles to create a computational model are presented and discussed on simple examples, especially the consideration of faults. The entire method is discussed in an example which was recently carried out for the tender design of a major tunnel project in Austria.

2 GEOLOGICAL INVESTIGATIONS AND MODELLING

Site investigation for an underground structure is a difficult task which in many cases is underestimated. The overall goal of site investigation is to develop a consistent ground model including all geotechnical and hydraulic aspects relevant for the underground structure as the basis for the construction contract. In all phases of the design this final goal should be kept in mind. Realistic modelling of rock mass geometry and mechanical behaviour of rock masses depends on the understanding of geological processes and their complex interactions such as tectonic deformation, weathering and morphogenesis (Steidl 2003). Suitable investigation concepts call for methods in accordance with the project and design stage. They include desk studies, field mapping, and subsurface investigations (Riedmüller 1998, Riedmüller & Schubert 1999, Steidl et al. 2001, Riedmüller & Schubert 2001, Goricki et al. 2001).

The correct transfer of the geological-geotechnical rock mass model and its implications into a proper design is one of the most challenging tasks in an underground project. One of the main problems is the uncertainty inherent in the geological prediction. Other problems are the assessment of such factors as site management, workmanship and support effects or the care and experience applied in the updating of the geological model as excavation proceeds (Riedmüller 1997).

The correct way to consider the uncertainty is to describe the rock mass and its parameters by statistical distributions rather than by singular deterministic values. However, the basis for all further investigations is a sound geological model together with carefully investigated and reasonably selected rock mass parameters.

Geological singularities of the rock mass model such as faults, lithological boundaries, and aquifers have a significant influence on a tunnel project. The necessary data to describe singularities can be either measured in boreholes, outcrops, aerial and satellite images or estimated. These data form the basis for the statistical distributions applied in the proposed procedure. Results have to be reconsidered after each new investigation campaign. In addition to the lithological discrimination between rock types, significant differences within a certain rock type, e.g. spacing and frequency of discontinuity sets, or the unconfined compressive strength, can be described by frequency distributions. For example, the spacing of discontinuities can be obtained by applying scanline or window mapping techniques on selected outcrops or along drill cores. When a sufficiently large number of data cannot be obtained, the parameter distribution can be estimated by using statistical methods.

3 ROCK MASS CHARACTERISATION

3.1 General procedure

The geotechnical design, as part of the excavation design, serves as a basis for approval procedures, the determination of excavation classes and their distribution, and the determination of the excavation and support methods used on site (Schubert et al. 2001).

Figure 1 shows a basic procedure for the design of underground structures, consisting of four general steps to develop the geotechnical design, beginning with the determination of the Rock Mass Types and ending with the definition of excavation classes. This paper focuses on the first and second steps of the procedure. However, to provide an overview of the entire procedure the four steps are illustrated.

3.2 Determination of Rock Mass Types (RMT)

The first step starts with a description of the geological architecture and proceeds by defining geomechanically relevant key parameters for each ground type. The key parameter values and distributions are determined from available information and/or estimated with engineering and geological judgment. Values are constantly updated as pertinent information is obtained. Rock Mass Types are then defined according to the selected key parameters. The number of Rock Mass Types elaborated depends on the project specific geological conditions and on the stage of the design process.

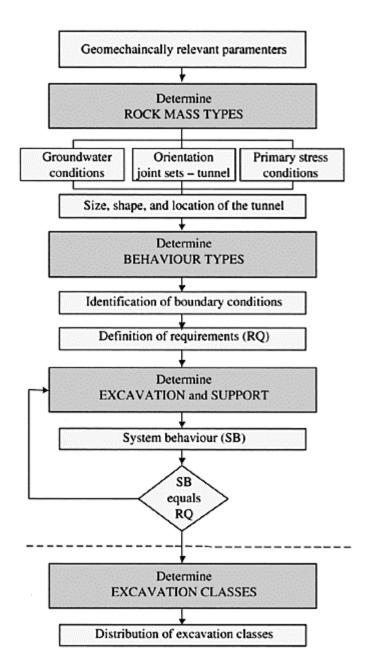


Figure 1. Flow chart of the basic procedure of excavation and support design for underground structures. Rock mass characterisation refers to

the first two main steps (from ÖGG 2001).

3.3 Determination of Rock Mass Behaviour Types (BT)

The second step involves evaluating the potential rock mass behaviours considering each Rock Mass Type and local Influencing Factors, including the relative orientation of relevant main discontinuity sets to the excavation, ground water conditions, primary stress situation, and the size, shape and location of the underground structure. This process results in the definition of project specific Behaviour Types.

Following the steps of figure 1, the rock mass behaviour is the response of a Rock Mass Type exposed to the acting Influencing Factors. It is determined on the final geometry of the excavation without considering support measures or excavation sequences. To evaluate this response, any suitable method is applicable. The authors predominantly used analytical models but under certain circumstances numerical or empirical methods are also reasonable (Thurner 2000, Schweiger et al. 2002).

The Rock Mass Behaviour Types form the basis for determining the excavation and support methods as well as assisting in evaluating monitoring data during the excavation.

The rock mass behaviour is the result of a failure of the rock mass surrounding the underground excavation. Each failure mechanism can be computed with appropriate analytical models and classified into defined Behaviour Types (Goricki 2003). The different failure mechanisms can be distinguished into gravity controlled failure of key blocks, gravity controlled failure of a highly fractured rock mass, stress induced failures such as shear failure, rock burst, spalling, buckling, and plastification, and other failure modes (ravelling ground, flowing ground and swelling ground).

3.4 Determination of the excavation and support

In the third step, different excavation and support measures are evaluated and acceptable methods are determined based on the defined Behaviour Types. The System Behaviour (SB) is a result of the interaction between the rock mass and its behaviour and the selected excavation and support schemes. The evaluated System Behaviour has to be compared to the defined requirements. If the System Behaviour does not comply with the requirements, the excavation and/or support scheme has to be modified until compliance is obtained.

3.5 Determination of excavation classes

In the final step of the design process the geotechnical design must be transformed into a cost and time estimate for the tender process. Excavation classes are defined based on the evaluation of the excavation and support measures, which is regulated for example in Austria with the standard ÖNORM B2203–1 by classifying round length and support measures. These excavation classes form a basis for compensation clauses in the tender documents.

The distribution of the expected Behaviour Types and the excavation classes along the alignment of the underground structure provides the basis for establishing the bill of quantities and the bid price during tender.

4 COMPUTATIONAL APPROACH

The characterisation procedure is performed by a computational model. This model follows the characterisation steps described above and promotes project-specific analyses and evaluations to cover the variety in geotechnical engineering and tunnelling. Probabilistic simulations are facilitated by this model.

4.1 General description

In general, the rock mass is subdivided into calculation segments to which Rock Mass Types and Influencing Factors are assigned. The rock mass behaviour is determined for each calculation segment using various analytical models to determine distinct failure mechanisms.

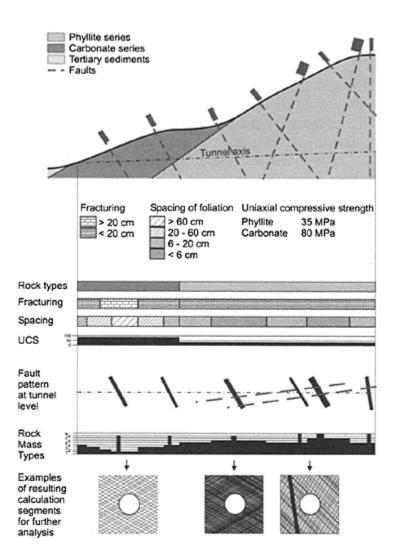
4.1.1 Calculation segments

The geological model is virtually cut into slices along the tunnel axis. These slices are called "calculation segments". They can have a constant or variable thickness and possess a virtual extent to avoid any boundary influence. These calculation segments are consecutively arranged along the tunnel axis and represent the discretised rock mass. Relevant properties and boundary conditions are assigned to the calculation segments.

The properties are assigned in terms of individual key parameters. Key parameters can be the rock type, the fracturing, and/or mechanical and hydraulic properties. The Rock Mass Type of the calculation segment results from the current combination of key parameters and their corresponding values.

In the next step the Influencing Factors (see 3.3) are assigned to the calculations segments. They represent the relevant boundary conditions of the calculation segment to the rock mass behaviour. The primary stress condition can be estimated from the corresponding overburden, and additionally from the genesis of the rock mass, the tectonic history, the location of the excavation within the rock mass, etc. The orientation of the main discontinuity sets relative to the excavation is based on both the field mapping and logging oriented drill cores.

The presence of ground water is usually estimated from a hydrogeological model. As mentioned before, for the determination of the rock mass behaviour only full face excavation is considered.



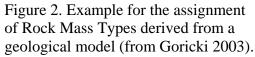


Figure 2 shows the described sequence of the assignment of rock mass properties with individual key parameters in a simple geological model. It consists of three lithological series which are a phyllite series, a carbonate series and tertiary sediments. Additionally, it contains the probable location of faults and their orientation. Relevant key parameters were identified which are the rock type and its unconfined compressive strength (UCS), the fracturing of the rock mass, the spacing of the foliation, and the presence of faults. These parameters are allocated to their probable location along the tunnel axis. Discretising the geological model into calculation segments and combining the

corresponding parameters result in one Rock Mass Type for each calculation segment. In the last row of Figure 2 three examples of resulting calculation segments are presented. These are a blocky rock mass (RMT 1), a faulted rock mass (RMT 8), and a foliated rock mass (RMT 7) with a fault near the tunnel wall.

4.1.2 Analysis of rock mass behaviour

The calculation segments are analysed with respect to the response of the current Rock Mass Type to tunnel excavation under the corresponding Influencing Factors. For the analysis different analytical models are simultaneously applied. Every model provides a physical value which describes an aspect of the rock mass response, e.g. displacements, depth of the failure zone, volume of overbreak, severity of rock burst, etc. Comparing these results to previously defined delimiting criteria various Behaviour Types can be distinguished. The most critical Behaviour Type is then assigned to the calculation segment.

For the estimation of displacement magnitudes and depth of failure zone the models from Hoek et al. (1995) or Feder & Arwanitakis (1976) are applied. The overbreak volume is assessed applying the principles of block theory (Goodman & Shi 1985) under consideration of the stress condition around the tunnel (Karzulovic 1988, Pötsch 2002).

The determination of rock mass behaviour in a fault zone is a crucial point in the application of the computational model. Tunnelling through a fault zone or heterogeneous rock mass is dominated by complex stress redistributions and irregular deformation characteristics due to the frequent changes of faulted material and competent rock blocks. In Figure 3 the changes in the stress magnitudes are depicted when the tunnel face approaches a softer rock mass. Stresses are attracted by the stiffer rock mass and considerably increase at the transition zone while stresses decrease in the softer zone. The magnitude of the stress redistribution depends on the stiffness differences between both zones. This fact has been extensively discussed (Schubert 1996, Schubert & Riedmüller 1997). Großauer (2001) proposed expressions based on numerical investigations for the changes of stress in a rock mass with frequent stiffness differences.

The behaviour is also influenced by the length of these zones. Displacements in a short soft zone embedded in a stiff rock mass do not develop to the values which would have been developed in an infinitely long soft zone. The stress increase in the stiff rock decreases with the length of the soft zone.

4.1.3 Deterministic evaluations

After assigning the most critical Behaviour Type to the calculation segments, the distribution of the Behaviour Types along the tunnel is obtained by adding up the length of calculation segments with the

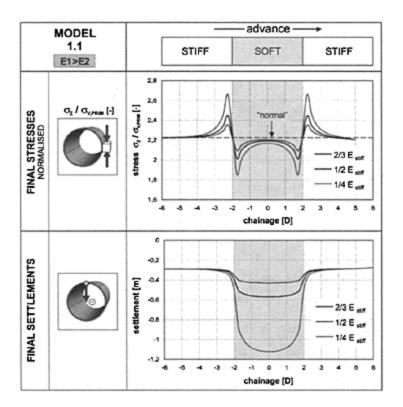


Figure 3. Influence of the variation of rock mass stiffness on secondary stress condition and settlements (from Großauer 2001).

same Behaviour Type. These evaluations provide one value for the length of each Behaviour Type. Since no spread or uncertainty for the input parameters is considered, the parameter values are critical.

4.2 Statistical Analytical Model (SAM)

The described computational model requires input parameters which usually cannot be provided as singular values with sufficient accuracy in rock engineering practice. Hence, every deterministic analysis performed with this method has a certain degree of uncertainty which cannot be quantified without further considerations. In this chapter a simple method to process uncertain input parameters in geological and geotechnical modelling is presented.

4.2.1 Probabilistic data processing

A probabilistic method is used to process distributed (uncertain) input parameters. The method uses the same computational model as described in Chapter 4.1. The difference is that the singular values of the input parameters are now replaced by statistical distributions. The results (e.g. lengths of different Behaviour Types along the tunnel axis) are also obtained in terms of statistical distributions. In the proposed method the integration of the output functions is performed with a Monte-Carlo-Simulation. In this simulation the computational model is consecutively calculated various times while in every calculation step (iteration) the input parameter values are varied according to their statistical distribution. The number of iterations must be sufficient to obtain a constant result. The output values are sampled and approximated by another statistical distribution using a bestfit procedure.

Since all results are statistical distributions, their spread can be analysed. This serves to assess the degree of uncertainty, to evaluate tendencies, or to design further investigations. It also can provide a basis for safety considerations or bid price calculations.

4.2.2 Probabilistic implementation of geological conditions

The three-dimensional geological model is used to statistically evaluate the desired input parameters. Usually the model is based on acquired data. Regions between the acquired data are extrapolated taking into consideration the consistency of the model with geological judgment. The model consists of an acquired (known) part and an extrapolated (uncertain) part.

From the acquired data several evaluations can be taken in order to obtain input functions for the probabilistic simulation. For instance, data about length and sequence of rock types in the drill cores, fracturing of the rock mass, correlations between weathering, depth and lithology, can be extracted and form the basis for input distributions. These data are analysed with respect to possible correlations. Additionally, Markov chains of sequences have to be determined. A Markov chain defines the probability of occurrence of the following element based on the type of the present element. Markov elements in this case are different rock types. An example is given in Chapter 5.4.3.

During the computational process the distributed input parameters and Markov chains are used to generate rock types and rock mass parameters along the tunnel axis. The described generation technique allows a probabilistic simulation of geological conditions using Monte-Carlo-Simulation techniques. During a Monte-Carlo-Simulation the rock mass model is newly generated in every iteration by the rules described above. The same process can be applied for the generation of faults. Since the parameters thickness, spacing and orientation of faults can be provided as statistical distributions, faults can be located along the tunnel axis. This results in a new fault pattern in every iteration. A detailed example is given in Chapter 5. Using this procedure the uncertainty of the geological prediction is taken into account with reasonable effort. Additionally, unfavorable geological conditions may arise which were not obvious before the simulation and the probability of their occurrence can be quantified.

4.2.3 Probabilistic consideration of geotechnical parameters

Once the computational model contains the geological situation, the geotechnical rock mass properties such as the friction angle, cohesion, or Young's modulus for the current iteration have to be assigned. This can be done using the distributed input parameters defined in the Rock Mass Types. This means that the parameter values of the Rock Mass Types also have to be provided as statistical distributions (Goricki et al. 2003).

Another way of introducing geotechnical parameters is to provide intact rock parameters for each rock type in terms of statistical distributions and assign them according to the current rock type of the calculation segment. The rock mass parameters are then determined taking into account the current fracturing and/or weathering, and the condition of the discontinuities. Based on these combinations the GSI (Hoek et al. 1995, Hoek & Brown 1997, Hoek 1999) can be estimated and rock mass parameters determined for the current calculation segment. The advantage of the second approach is its higher flexibility in the consideration of special rock mass features. For instance, the influence of discontinuity orientation or water presence on strength parameters can be consistently implemented. Furthermore, it allows considering the entire spread of the parameter values according to the determined distributions.

With the described computational model it is possible to take into account the uncertainties of the geological prediction and the spread of the geotechnical parameters. Using this model rock mass characterisation can be easily performed and its reliability quantified.

5 APPLICATION AND CASE STUDY

The presented characterisation procedure has been applied to many tunnel projects in different stages of design as well as during construction. This case study outlines the application of the procedure and the software tool for the tender design of Koralm Base Tunnel, a major tunnel project in Austria. The complete case study is based on the corresponding ground expertise (3G 2004).

5.1 Project description

The Koralm Base Tunnel will form part of the Austrian High Speed Railway Network. It links Styria with Carinthia by a planned 32.8 km double-tube with an overburden of about 1200 m at its maximum (Fig. 4). Due to the extension of the High Speed Railway Network where the tunnel is one of the core pieces, the travel time from Vienna to Klagenfurt can be reduced from four and a half to three hours (Vavrovsky et al. 2001).

Within the scope of the investigation campaign, three exploratory tunnels are going to be constructed.

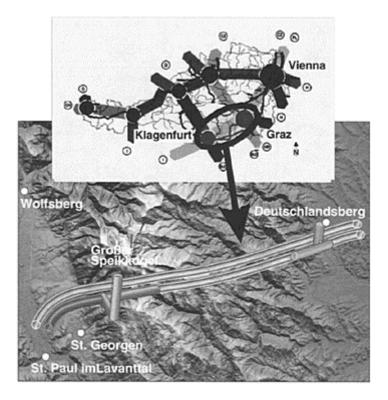


Figure 4. Overview of the Koralm tunnel project including the exploratory tunnels "Mitterpichling", "Paierdorf", and "Leibenfeld" (from left to right) and the shaft "Paierdorf" (left) and "Leibenfeld" (right).

These tunnels are the exploratory tunnels "Mitterpichling", and "Paierdorf" on the Carinthian side, and the exploratory tunnel "Leibenfeld" on the Styrian side of the Koralm massive (Fig. 4). The length of the "Paierdorf" tunnel is about 5.5 km and the maximum overburden is about 700 m. It starts in a depth of about 160 m. The access is provided by the shaft "Paierdorf" with a diameter of 9 m. It is a very challenging project since it explores for a long stretch the conditions of Lavanttal fault zone. In this chapter the investigation and characterisation process is exclusively described for the "Paierdorf" exploratory tunnel which was performed during the tender process, and focuses on the approximated 2 km of the fault zone.

5.2 Geological overview

The Koralm mountain range is part of the Koriden unit within the Middle-Austroalpine nappe complex of the Eastern Alps. The majority of the Koralm tunnel is located within a polymetamorphe crystalline basement, mainly consisting of different types of gneiss, mica schist, and secondary units including quartzite, amphibolite, eklogite and marble. The Koralm is bounded on both sides by young extensional brittle fault systems. The most important fault system is the Lavanttal fault system located at the western hill slope of the Koralm. This NNW-SSE striking fault system is supposed to be active and generated the Lavanttal valley, a pull-apart basin with a depth of more than 1000 m. This basin is, for the most part, filled with tertiary, fine-grained river and marine sediments.

5.3 Geological model for the Paierdorf exploratory tunnel

On the basis of the results of the engineering geological investigations, a threedimensional geological rock mass model of the Koralm area was developed.

The investigations focused on

- definition and description of the relevant lithological sequences,
- detailed engineering geological characterisation of the rock mass,
- identification and characterisation of fault zones including the evaluation of fault kinematics and paleo-stress analysis, and
- evaluation of the groundwater situation.

Special emphasis was given to the tectonic interpretation.

The metamorphic bedrock in the project area is characterised by frequent lithological variations and smooth transitions from one rock type to the other. Field mapping and core logging results reveal that these changes occur at scales from a few decimetres to some tens of metres. Larger lithological sequences were distinguished during field investigations. The following lithological sequences were distinguished for the metamorphic bedrock:

- gneiss sequence,
- mica schist sequence,
- marble sequence,
- amphibolite sequence,
- cataclastic rocks.

The overlying tertiary sediments were differentiated according to their grain size and strength.

The geological conditions for the Paierdorf exploratory tunnel (Fig. 5) can be very roughly subdivided into three sections. The dominating geological structure, which is most critical for tunnelling, is the Lavanttal fault system at the lower western hill slope of the Koralm. The entire section is characterised by a heterogeneous composition (tectonic melange) of weak fault rocks and sound parent rocks (Medley 2001). The faults strike NW-SE and steeply dip towards the Lavanttal tertiary basin. They are oblique slip to dextral transform faults. The entire section is characterised by a complex groundwater situation of aquifers and aquicludes. The water inflows may exceed some hundreds of litres per second.

The Lavanttal fault system generated the Lavanttal tertiary basin which includes finegrained sedimentary rocks with very low rock strength. These rocks

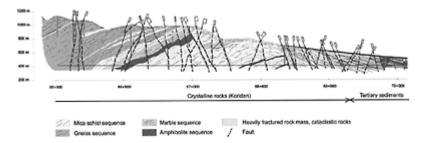


Figure 5. Sketch of the geological longitudinal section of the Paierdorf exploratory tunnel.

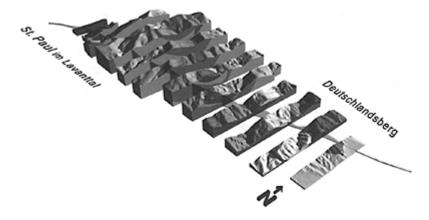


Figure 6. Three-dimensional geological model of the project area. The Paierdorf exploratory tunnel is located on the western hill slope of the massive.

are sensitive to water with a high potential of slaking. Minor water inflows are locally expected.

The upper western hill slope of the Koralm is characterised by a large-scale syncline structure trending approximately WNW-ESE. The rocks generally are thickly bedded to massive and slightly jointed. Weathering phenomena including discontinuity surfaces, alteration of rock material resulting in rock strength reduction, open joints, etc. were generally encountered up to a depth of about 200 metres. The field investigation indicates that there is a potential for encountering zones with elevated rock mass temperatures.

Temperatures of approximately 25 to 30°C are expected. Within that section the tunnel is expected to have dry to damp conditions. Higher water inflow rates of several tens of litres per second are expected at fault zones or fracture zones.

5.4 Computational model and input parameters

Based on the results of site investigations, a spatial rock mass model was developed (Fig. 6) and the expected lithology, geological architecture, fault zones and ground water situation was characterised. Due to the geological situation a central part of the characterisation process focused on the investigation and modelling of the behaviour of faults in addition to the behaviour of the unfaulted rock mass.

5.4.1 Modelling the basic geological architecture

During the evaluation of the investigation results, four lithological sequences could be distinguished. The names of the sequences correspond to the dominating rock type within the sequences (Table 1). The location of the lithological sequences in the geological model has been determined with reasonable accuracy.

Lithological sequence	Rock types	Percentage
Gneiss	Gneiss	93
	Mica schist	5
	Amphibolite	
	Marble	2
Mica schist	Gneiss	10
	Mica schist	86
	Amphibolite	2
	Marble	2
Marble	Gneiss	2
	Mica schist	12
	Amphibolite	21
	Marble	65
Amphibolite	Gneiss	3
	Mica schist	10
	Amphibolite	83
	Marble	4

Table 1. Contribution of the determined rock types to the lithological sequences in percent.

Therefore, these sequences have been considered to be fixed (Fig. 5 & Fig. 6). The distribution of the rock types within the sequences has been probabilistically modelled using the contributions of each rock type as shown in Table 1. The length of each zone of a unique rock type has been derived from evaluations of drill cores and related to the tunnel location and orientation.

Another important input parameter is the fracturing of the rock mass. It is the classification of the maximum dimension of a piece of a drill core into categories (Fig. 7 & Fig. 8). It is consecutively determined along the drill axis for all core pieces. It serves as one of the major input parameter for the determination of rock mass properties. Correlations between the

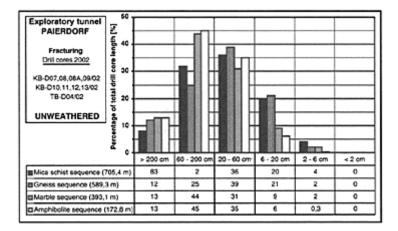
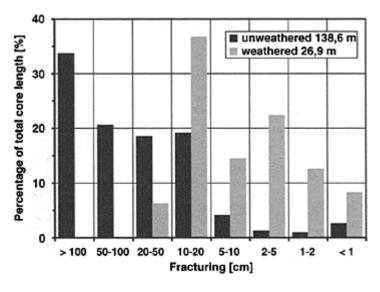
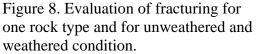


Figure 7. Evaluation of fracturing for the four identified rock types for unweathered condition.

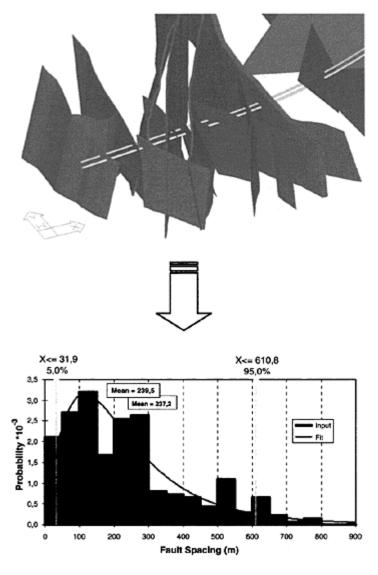


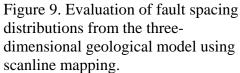


fracturing and rock types, and fracturing and weathering, respectively, have been taken into account.

Figure 7 shows the distributions of the fracturing for the four identified rock types in unweathered condition evaluated from the logged drill cores. The evaluations were made from 9 core drillings with an overall length of about 1800 m. Figure 8 shows the results of a statistical evaluation of fracturing and weathering data obtained from rock cores in a 166 m deep borehole. The diagram clearly shows a changing distribution for weathered and unweathered rock mass.

Zones of fracturing were generated according to the corresponding rock type and weathering with respect to the determined distributions. It is evident that changes of fracturing occur more often than changes in rock types. In the computational process this fact was considered by subdividing the rock type regions into zones of different fracturing. This promotes a rock mass model with a higher diversification. The rock types and its fracturing are subsequently superimposed in the computational model for further analyses.





5.4.2 Modelling faults as singular elements in the rock mass

One approach to model faults in a computational model is to acquire their thickness, spacing or frequency, and orientation relative to the tunnel axis. Based on these

parameters faults can be generated probabilistically. This method is reasonable when the faults are thin compared to the unfaulted rock mass, i.e. when the fault frequency is relatively low.

With this approach various unfavorable effects of faults due to their location can be taken into account. For instance, the quantity and probability of faults striking subparallel to the tunnel which may cause massive overbreak or highly overstressed tunnel walls can be consistently estimated. The previously generated geological model is adjusted to the newly generated fault pattern. The interdependence between adjacent faults can also be determined. The distributions of fault thickness were determined from the results of core logging and the evaluation of optical scanner measurements. The frequency of major fault zones (thickness greater than 5 meters) was derived from the results of detailed geological field mapping by applying scanline mapping techniques (Fig. 9).

5.4.3 Modelling a rock mass with a high fault frequency

In a rock mass with a high fault frequency, the interdependence between faults is dominating. Rock mass behaviour is controlled by stress redistributions due to stiffness contrasts in the rock mass. To account for this effect in a heterogeneous rock mass, another approach was applied.

The drill cores were evaluated with respect to the stiffness of the core pieces. For tunnelling the stiffness of the rock type and the fracturing has a major influence on the rock mass stiffness. This evaluation has focused on rock type and fracturing. Figure 10 shows the results of the evaluations of the drill cores of the project area. The bars represent the stiffness magnitudes of the corresponding cores. Stiff, moderately stiff and soft zones were distinguished.

On the left hand side of the bars the determined stiffness magnitude of the core pieces according to the log is shown, whereas on the right hand side the resulting stiffness magnitude of the rock mass relevant for tunnelling is shown. The sequence of these sampled stiffness zones is analysed with respect to the distribution of their lengths and Markov chain (Table 2).

In the probabilistic simulation the sequence of stiffness zones obeys the values shown in Table 2. This means that, if the current zone is defined as "stiff" the following zone is "soft" with a probability of 0.22 or "moderate" with a probability of 0.78. Using this query the sequence is generated. Once the sequence is clear, the lengths are assigned to each zone. The values are taken from density functions which are provided for each stiffness zone type (Fig. 11). The generated sequences are added to the computational model. This computational model therefore represents a geological model with the same parameters as the real rock mass determined with drill cores.

5.4.4 Determination and assignment of geotechnical parameters and influencing factors

Mechanical parameters for the various rock types were determined with an extensive number of samples on laboratory test including uniaxial and triaxial compression tests, direct shear tests, Brazilian tests, and direct tension tests. From these tests typical parameters such as the unconfined compressive strength (UCS), mi value, cohesion, friction angle, the Young's modulus, etc., were determined. Due to the extensive number of tests it was possible to statistically evaluate the results and provide statistical distributions as input parameters for the intact rock.

In the probabilistic simulation the parameters are assigned to each calculation segment according to the probability function for the mechanical parameter of the corresponding rock type. Correlations between parameters or other boundary conditions have to be considered. For instance, soft zones have to be linked with a highly fractured rock mass or a rock type with a low Young's modulus.

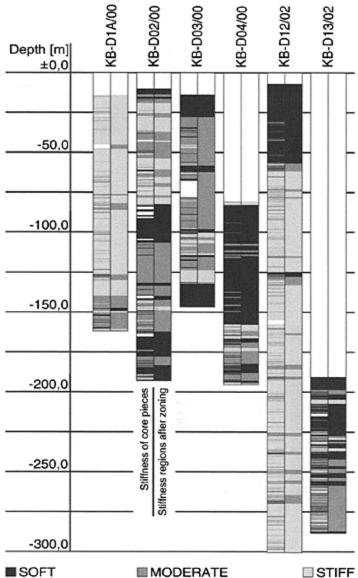
Based on the parameter combination of a calculation segment a GSI has been assigned to determine the rock mass properties. The GSI was determined based on a combination of fracturing, weathering, and discontinuity condition.

The Influencing Factors were assigned according to the geological model. The magnitude and orientation of the primary stress was derived from the overburden, the tectonic history, and hydraulic fracturing in situ tests. This led to the estimation of high lateral stress in the part of the tunnel with high overburden, and, to a moderate lateral stress level in the western periphery of the Koralm (Lavanttal) due to the extensional tectonic history. Other influencing factors were analogously assigned to the approach given in previous chapters.

5.2 Evaluation of Rock Mass Behaviour

With the computational model and the described input parameters, a Monte-Carlo-Simulation with 5000 iterations was performed. The calculation segments were evaluated in order to determine the most unfavorable rock mass behaviour using the calculation models described in chapter 4.1.2. The rock mass behaviours were classified, based on predefined delimiting criteria, into Behaviour Types (Table 3).

Figure 12 shows the mean values of the lengths of Behaviour Types expected in the region of high fault frequency. It is evident that about 65% represent rock mass behaviour with stress induced failure (BT 3, 4 and 11), with about 39% in BT 11. Figure 13 shows a typical result for the relative lengths of BT 11 in terms of a probability density function. It includes the definition of the probability density function (BetaGeneral), the mean value, and the 15%- and 85% quantile. It reveals the inherent scatter in the results of a probabilistic simulation due to the scatter and/or uncertainty of the input parameters.



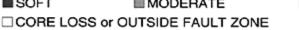


Figure 10. Evaluation of the drill cores in the project area with respect to stiffness contrasts.

Zone followed by	Soft	Moderate	Stiff	
Stiff	22	78	_	
Moderate	39	_	61	
Soft	_	83	17	

Table 2. Markov chain of the stiffness sequence derived from Figure 10. The values represent the probability of occurrence in percent.

Negative-Exponential-Distribution

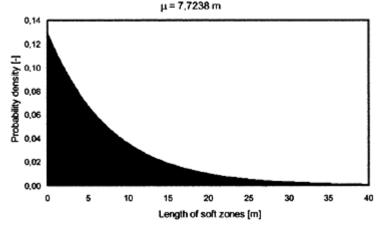


Figure 11. Probability density function for the determination of the length of a soft zone derived from Figure 10.

Based on this analysis, excavation and support can be designed and risk analyses can be performed.

Figure 14 gives an overview of the results for a region with a high and a low fault frequency. It includes sketches of the failure mechanisms and the

Table 3. Behaviour Types defined in ÖGG (2001). A detailed description is given in Goricki (2003).

Numbe	er Behaviour Type
1	Stable
2	Discontinuity controlled block failure
3	Shallow stress induced failure
4	Deep seated stress induced failure

5	Rock burst
6	Buckling failure
7	Shear failure under low confining pressure
8	Ravelling ground
9	Flowing ground
10	Swelling
11	Frequently changing behaviour

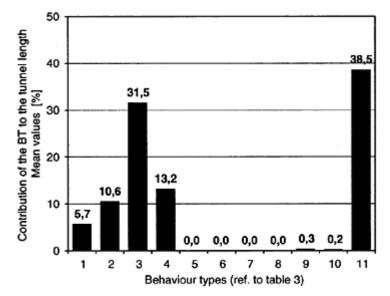


Figure 12. Mean values of predicted Behaviour Types for a region with high fault frequency. The numbers refer to Table 3.

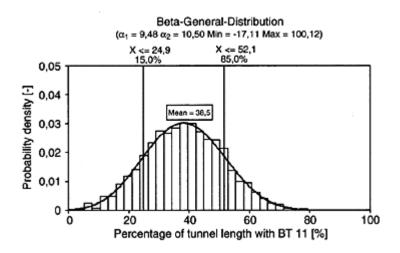


Figure 13. Probability density function of the relative lengths of BT 11 in the region of high fault frequency. The mean value is about 39% whereas the 15%-quantile is about 25% and the 85%-quantile is about 52%.

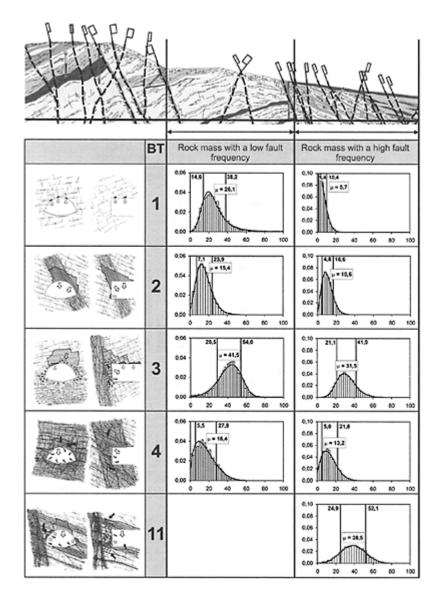


Figure 14. Overview of the results of the determination of the Behaviour Types for two regions of the Paierdorf exploratory tunnel including corresponding sketches of failure mechanisms and probability density

functions of the length of occurrence for each BT.

corresponding probability density functions (compare also Figure 13).

According to the Austrian Guideline (ÖGG 2001) the obtained Behaviour Types have to be verbally and graphically described. The descriptions must include specifications about the involved Rock Mass Types, the primary stress condition, orientation of the main discontinuity sets relative to the excavation, the ground water situation, the excavation shape, and the dominating failure mechanisms, behaviour at excavation, breakage, long-term behaviour and the estimation of the final displacement magnitude.

6 SUMMARY AND CONCLUSION

A procedure has been presented to consistently determine the rock mass behaviour in the geomechanical design process. The requirements on the geological investigation and the applied methods have been discussed. A general procedure for rock mass characterisation and determination of excavation and support has been presented. It includes the determination of Rock Mass Types, Behaviour Types, the System Behaviour, and excavation classes. Subsequently a computational model has been described which follows the proposed general procedure in the determination of Behaviour Types. The computational model promotes probabilistic simulations including the consideration of uncertainties in the geological model and geotechnical parameters. A case study of a rock mass characterisation for the tender design of a tunnel has been presented. The establishment of a geological model, the derivation of statistical input parameters and the quality and quantity of the results have been extensively described.

The computational model provides flexible and project-specific evaluation possibilities and can be easily extended to determine excavation and support on a probabilistic basis which has been described by Großauer et al. (2003). Furthermore, it is a simple method to determine the risk due to uncertainties in the rock mass. This serves as a basis for risk analyses and cost estimations (Goricki et al. 2002a, Goricki et al. 2002b).

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3 Modelling

Rock mass characterization for numerical modelling of ground stability control in mining

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ABSTRACT: Following a brief review on the currently widely used numerical approaches in mining geomechanics study, rock mass parameters along with stress characters that significantly affect modelling results are analysed through examples using different numerical methods. The sensitivity of mining geomechanics modelling results to input parameters and applied rock mass failure criteria are tested through different numerical approaches both 2D and 3D. A comparison is also made of numerical modelling between using rock mass parameters as input with empirical failure criteria in continuum modelling methods and discontinuum modelling methods that mimics the behaviour of both intact rock and rock mass discontinuities. Some cautions are raised in order to correctly use and interpret mining geomechanics modelling results in mining practice.

1 INSTRUCTIONS

Numerical modelling of ground stability in mining is one of the major applications of rock mechanics modelling which has developed for the design of rock engineering structures. Hudson (J.A.Hudson, 2001) categorised numerous rock mechanics modelling into four basic methods. Of the four methods, Method A is based on previous design experience. Method B is based on analytical analysis with simplified constitutive model. Method C and D are based on numerical modelling with input from site investigations for rock mass characterisation. With the accumulative experience on the correlations between the rock mass reality and the outcomes of numerical modelling and improved modelling tools, a clear trend exists that mining engineering design in relation to rock mechanics relies more and more on Methods C and D.

Because rock is a natural geological material with its physical and engineering properties having to be established rather than being defined through a manufacturing process (L.Jing, 2003), the complex combination of rock mass constituents and its long history of formation make it a difficult material for mathematical representation in numerical modelling. The difficulties are basically reflected in two aspects. One is in developing constitutive models representing the true behaviour of rock mass and its engineering structures, the other is quantitative characterisation of a rock mass for computational analysis using the constitutive models. In coping with these difficulties to achieve the best numerical representation of a physical rock engineering problem, three categories of numerical methods for rock mechanics problems have been developed and commonly used as follows:

- Continuum methods: Finite Element Method (FEM), Boundary Element Method (BEM), and Finite Difference Method (FDM).
- Discontinuum methods: Discrete Element Method (DEM) and Discrete Fracture Network (DFN) methods.
- Hybrid continuum/discontinuum models: Hybrid FEM/BEM, Hybrid DEM/DEM, Hybrid FEM/ DEM, and other hybrid models.

From the mining application point of view, common intersects lie in the latter aspect in achieving the best quantitative characterisation of a site specific rock mass and a good prediction of excavation behaviour in mining practice. In general numerical modelling of mining rock mechanics, the following rock characterisation problems would be encountered:

- Rock mechanical properties from lab testing work vs. rock mass properties on a large scale;
- Rock mass classification and quantification;
- In-situ stress regime;
- Rock mass failure criteria.

It is relatively simple to characterise a particular rock mass in a particular location of one specific mine site by using certain type of rock property characterization. But it is subtle to select an appropriate numerical modelling tool to analyse the particular rock mechanics problem. Therefore, the question of whether a numerical model is creditable in capturing the rock mass reality relates to both numerical modelling methods employed and associated rock property characterisations. This paper discusses these issues through practical examples in mining rock mechanics practices.

2 REPRESENTATIVENESS OF INTACT ROCK STRENGTH PROPERTIES FROM LAB TESTING TO ROCK MASS ON A LARGER SCALE

It is straightforward to determine a set of mechanical properties for intact rock samples in an established rock mechanics laboratory. However, there are two factors that have major impact on the intact rock UCS value to be used for rock mass strength determination for numerical modelling. One factor is the sample size dependence of intact rock UCS. The influence of sample size upon rock strength has been widely discussed in geotechnical literature and it is generally assumed that there is a significant reduction in strength with increasing sample size. Based upon an analysis of published data, Hoek and Brown (1980) have suggested that the uniaxial compressive strength σ_{cd} of a rock specimen with a diameter of d mm is related to the uniaxial compressive strength σ_{c50} of a 50 mm diameter sample by the following relationship:

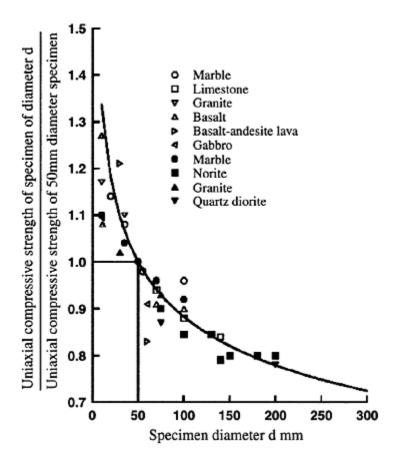
$$\sigma_{cd} = \sigma_{c50} \left(\frac{50}{d}\right)^{0.18} \tag{1}$$

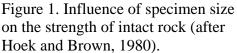
This relationship, together with the data upon which it was based, is illustrated in Figure 1. According to Hoek and Brown, the reduction in strength is due to the greater opportunity for failure through and around grains, the 'building blocks' of the intact rock, as more and more of these grains are included in the test sample. Eventually, when a sufficiently large number of grains are included in the sample, the strength reaches a constant value.

The second factor is the wide range variation of UCS values of core samples that are of the same rock type and drilled from the same location. For example, an average UCS of 100 MPa may come from a set of three samples. Each of which gives 80, 100 and 120 MPa, respectively. It is a common practice to use the mean value in determining rock mass strength. However, the true value could be either close to the lower boundary or close to the upper boundary if more samples were tested.

In demonstrating the significance of intact rock strength in numerical stress modelling, a series of computational runs were conducted to study the stability of an underground intersection and examine requirements for rock reinforcement.

A three way junction comprised of a 6 m×6 m decline and a 7 m×6 m chamber to be developed at a depth of 500 m and 800 m below ground surface, as shown in Figure 2, was analysed using Map3D (T.Wiles, 2003). The average Western Australian underground mine stress regime was assumed. The rock mass properties used are as follows:





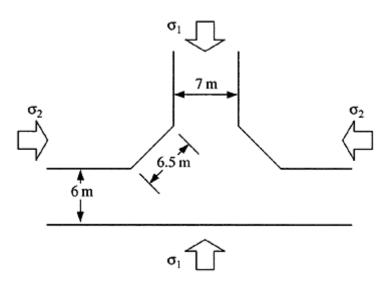


Figure 2. Three way junction for MAP3D modelling.

- Intact rock UCS: 50~150 MPa;
- Intact rock material constant: m_i=20;
- Rock mass constant: GSI=70, m_b=6.85, s=0.0357;

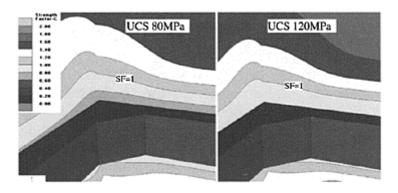


Figure 3. Overstressed zone in the back of an intersection (at a mining depth of 500 m).

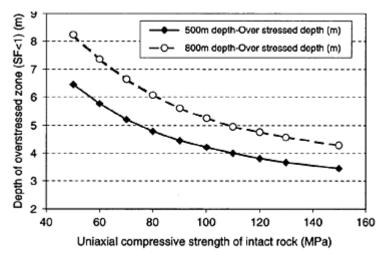


Figure 4. Height of overstressed zone with intact rock strength in the intersection back at 500 m and 800 m depth.

- Rock mass strength: 20~60 MPa;
- In-situ virgin stress regime:

 σ_1 =0.066 Depth+5.46 (MPa) σ_2 =0.041 Depth+2.63 (MPa) σ_3 =0.027 Depth+0.78 (MPa)

Using the Hoek-Brown criterion, the modelling produces strength factor (SF) which is the factor of safety of the rock mass as shown in Figure 3 and Figure 4. With the intact rock strength varying from 50 MPa to 150 MPa and the same set of assumed rock mass constants, the rock mass strength varies from 20 to 60 MPa according to Hoek-Brown criterion. The depth of over stressed surrounding rock in the intersection back varies from 6.5 m to 3.4 m and 8.3 m to 4.3 m at a mining depth of 500 m and 800 m respectively. While these numbers give a good guideline for designing the length of cable bolts for reinforcing intersections, they also show a high level of sensitivity of modelling results to the intact rock UCS value that was used in determining rock mass strength.

3 BACK ANALYSIS WITH REFERENCE TO IDENTIFIED STRESS INDUCED FAILURE

Numerical stress modelling in many cases is used for strategic design of new mining stages of an existing mine, where rock mass behaviour in the following forms is observed or measured:

• Displacement of openings or excavations have been measured;

- Ground support performance has been evaluated;
- Stope wall/back stability has been recorded with failure scale assessed;
- Seismic activities have been recorded.

These quantitative or qualitative measurements/ records give a very good indication of rock mass behaviour with respect to mining induced stress, excavation dimensions/layout, mining sequence and ground support measures. They can be used for quantitative generalisation of the rock mass properties through back analysis on either a large scale (such as multiple level mining operations) or a relatively small scale (tunnel stability for instance).

On a relatively small scale, the properties of rock mass and in-situ stress around a single excavation with recordable deformational behaviour can be back analysed through trial-and-error using the forward analysis software package. A set of experimental input parameters that produce modelling results in good agreement with the actual measurement of the excavation can be used for subsequent forward analysis.

However, mining geomechanics modelling generally involves multiple openings/excavations where quantitative measurement of the rock mass around the openings is unavailable either due to access difficulties or for practical reasons. Therefore, qualitative back analysis is normally a practical approach to achieving a set of site specific rock mass parameters.

As an example, a multiple level underground mining operation with five existing mining levels had observed a number of stope failures while ore extraction was taking place from level 1 down to level 5. The following conclusions in general were drawn from observations and history records of stope and pillar performance over years of mining from level 1 to 5:

- The sill pillars with a height of less than 5 m were typically stressed to failure and could involve substantial collapse;
- Rock falls usually occurred in stopes where there was a void in the immediate upper level and no sill pillars or only small sized sill pillars existed;
- A rib pillar of approximate 5 m appeared to be stable and would remain in place with minor yielding.

In addition to the general stope and pillar performance, an unstable pillar in a level 4 intersection, where vertical cracks caused by mining induced stress was visible, was used for cross checking the calibrated input parameters including rock mass strength and modulus.

Using trial-and-error method, the calibrated EXAMINE3D (RocScience, 2003) model for the existing underground mine produced results shown in Figure 5, where the unstable areas are identified with a SF<1. By comparing with the history records of stope performance, it was deemed that the calibrated model and input parameter produced stope and pillar behaviour that is in good agreement with the general trend of observed stope and pillar performance.

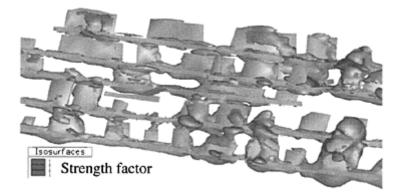


Figure 5. 3D strength factor of open stopes.

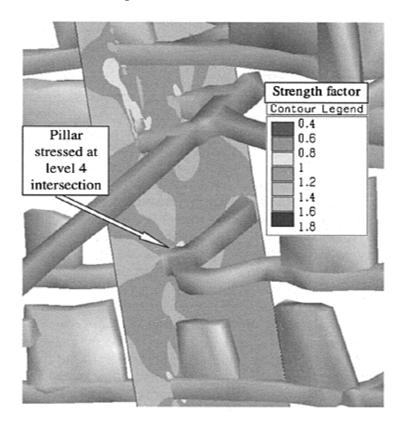


Figure 6. Good match of pillar stress in modelling with reality.

With the calibrated input parameter, the modelling results in Figure 6 shows that the pillar at the level 4 intersection has a SF of 0.8~1.2. This indicates that the pillar is within stress equilibrium to failure. These features match the site observation very well.

With the calibrated rock mass properties, forward analysis modelling in predicting deeper level mining induced stress conditions and resultant excavation stability can be conducted with reasonable creditability.

4 FORWARD ANALYSIS FOR CASES WITH IMPRACTICAL BACK ANALYSIS

In many cases, there is no stress induced rockmass failure that has been observed and could be used for back analysis. For example, at the design stage of an underground mine that is below a existing shallow open pit. No stress related rock mass failure has ever occurred in the fresh rock section of the open pit walls. The only information that could be deduced from the stable standing open pit is that the rock mass strength factor is greater than 1. But the upper boundary of SF is unknown. Though a low boundary of rock mass strength can be found out by adjusting the input UCS value to such a point that the modelled slope just starts showing areas with SF<1, the results from forward analysis using this UCS value is obviously conservative and are likely to be unrealistic. Under this circumstance, the rock mass properties may be gained through in pit geotechnical mapping and Bieniawski's Rock Mass Rating (RMR) system (Z.T.Bieniawski, 1984) in conjunction with Hoek-Brown criterion (Hoek, Carlos and Brent, 2002). This approach involves the following procedures:

- Using the five rock mass classes from RMR rock mass classification and six classes of rock mass structural classification to find out the Geological Strength Index (GSI) (Hoek, Wood and Shah, 1992);
- Finding the intact rock material constant according to rock type;
- Estimating the rock mass disturbance factor D according to the application, excavation method and blast impact;
- Calculating the Hoek-Brown's three rock mass material constants according to the following equations:

$$m_b = m_i \exp\left(\frac{GSI - 100}{28 - 14D}\right) \tag{2}$$

$$s = \exp\left(\frac{GSI - 100}{9 - 3D}\right) \tag{3}$$

$$a = \frac{1}{2} + \frac{1}{6} \left(e^{-g_{3}i/15} - e^{-20/3} \right) \tag{4}$$

An approximate Mohr-Coulomb fit may also be achieved using the equations derived from HoekBrown criterion as follows (Hoek, Carlos and Brent, 2002).

$$\tau = c' + \sigma \tan \phi'$$

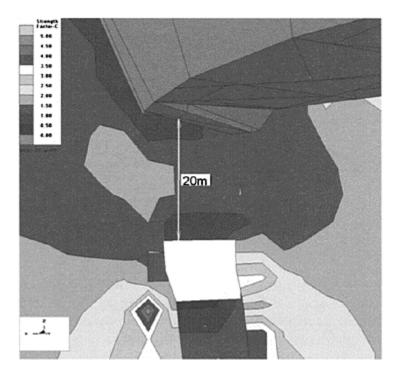


Figure 7. Forward analysis for 20 m crown pillar stability with rock mass input from geotechnical mapping and Hoek-Brown criterion.

(5)

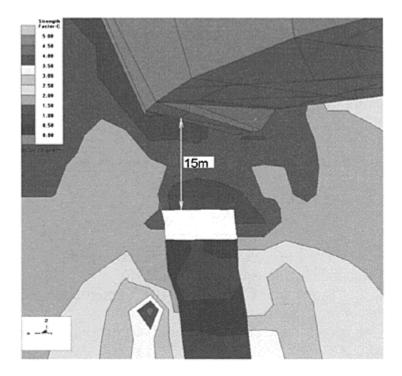


Figure 8. Forward analysis for 15 m crown pillar stability with rock mass input from geotechnical mapping and Hoek-Brown criterion.

where c' and
$$\Phi'$$
 are expressed in the following equations:

$$\phi' = \sin^{-1} \left[\frac{6am_b (s + m_b \sigma_{3n})^{a-1}}{2(1+a)(2+a) + 6am_b (s + m_b \sigma_{3n})^{a-1}} \right]$$
(6)

$$c' = \frac{\sigma_{el} \left[(1+2a)s + (1-a)m_b \sigma_{3n} \right] (s + m_b \sigma_{3n})^{a-1}}{(1+a)(2+a)\sqrt{1 + (6am_b (s + m_b \sigma_{3n})^{a-1})} / ((1+a)(2+a))}$$
(7)

5 SENSITIVITY TO IN-SITU STRESS ORIENTATION AND MAGNITUDE

As discussed above, one of the two main factors dominating the stability of rock mass around an opening is in-situ stress and mining induced stress. In considering the rock mass stress, an emphasis is normally placed on the magnitude of principal stress with less attention being paid to their orientations. One of the main reasons is a lack of reference for conducting a validation of the orientation. As a matter of fact, the stress orientations are as critical as the magnitude. In demonstrating the importance of in-situ stress orientation, the following exercise was carried out with Examine3D in studying potential stope failure.

A tabular ore body with an average height of 40 m and a strike length of 195 m lies at an average depth of 200 m below ground surface. The host rock has an uniaxial compressive strength of 178 MPa with rock mass constants of 8.56 and 0.0357 for m_b and s, respectively, while applying the Hoek-Brown failure criterion. Due to the relatively shallow mining depth and none precedent stress induced rock mass failure, no in-situ stress measurement had been carried out. In assessing potential post mining stope failure that could occur in the upper levels and have impacts on the active mining stopes in the lower levels, the magnitude of principal stresses were estimated according to the simplified general Western Australian underground mine stress regime, ie. the minor principal stress is vertical and caused by gravity, the major and intermediate principal stresses are horizontal and equal to 3 and 1.5 times the minor principal stress, respectively.

Because the stress orientation has never been proved by site specific measurement, two sets of stress orientations were modeled using Examine3D, i.e. north-south major principal stress and east-west major principal stress. The results corresponding to each of the two sets of principal stress orientations are presented in Figure 9, where the dark areas are unstable zones (SF<1).

Clearly, the east-west major principal stress regime would potentially result in some minor post mining hanging wall failure, particularly in the upper levels

As an example, an existing open pit with a depth of 250 m and proposed underground ore extraction is modeled through Map3D in studying an optimal stable crown pillar height. With input parameters from geotechnical mapping and application of the generalised Hoek-Brown criterion, a stable crown pillar was determined such that at its minimal height there exists a core area in the pillar with a strength factor greater than 1. A comparison shows that 15 m is the optimum height while 20 m is an excessive height.

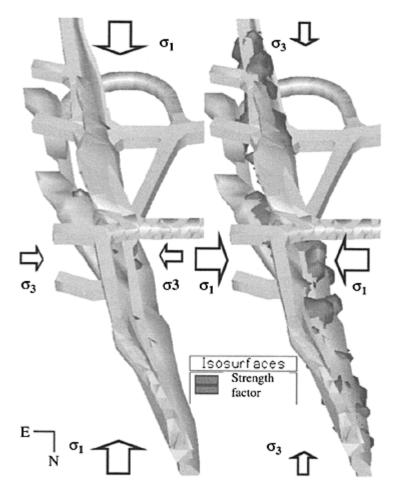


Figure 9. A comparison of stope stability with two sets of in-situ stress orientations (in plan view).

and cause possible rock fall into the active mining stopes in the lower levels and subsequent higher level of dilution. In contrast, the north-south major principal stress regime would almost produce no stope wall failure.

The example shows that a difference of input stress orientation would result in completely different assessment on the stability of open stopes. With the true stress orientation unknown, strategic stope design becomes a difficulty. A conservative approach towards this matter would be choosing the worse case scenario as a base for design. But the consequence of doing so would be an economic compromise because the true stress orien tation might not be the worse case scenario.

6 APPROPRIATE USE OF ROCK MASSFAILURE CRITERIA— HOEK-BROWN VS. MOHR-COULOMB

Rock mass failure criteria are important components of constitutive relations and are usually used to determine yield surfaces and/or plastic potential functions in a plasticity model. However, care must be taken when choosing a criterion for a specific application as a differentiation might exist between results produced from different criteria for the same problem in concern. For the widely adopted rock mass failure criteria,

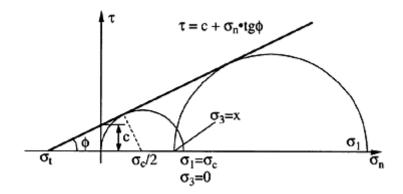


Figure 10. Linear Mohr-Coulomb failure criterion.

Hoek-Brown and Mohr-Coulomb criteria, a difference of applicability exists depending on the general mechanical nature of the materials analysed. This is mainly due to the tremendous difference between the tensile strength/UCS ratio reflected in these two criteria. As shown in Figure 10, the linear Mohr-Coulomb (Sheorey, 1997) failure criterion gives

$$\frac{\sigma_c}{2} = (\sigma_t + \frac{\sigma_c}{2})\sin\phi, \text{ ie. } \frac{\sigma_t}{\sigma_c} = \frac{1 - \sin\phi}{2\sin\phi}$$
(8)

where σ_t and σ_c are tensile strength and uniaxial compressive strength respectively, Φ is internal friction angle.

Simply, σ_t / σ_c is a function of material internal friction angle and can be graphically presented by the solid curve in Figure 11.

However, the Hoek-Brown criterion gives

$$\frac{\sigma_i}{\sigma_c} = \frac{s\sigma_{ci}}{m_b \bullet \sigma_{ci} s^a}$$
(9)

where s and m_b are rock mass constants and can be calculated by the equations (2) and (3)

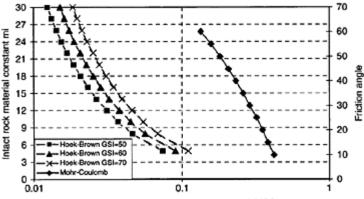
 σ_{ci} is the uniaxial compressive strength for the intact rock material.

a is rock material constant and can be generalized to 0.5.

Substituting equation (1) and (2) to (8) without considering blasting disturbance factor gives

$$\frac{\sigma_i}{\sigma_c} = \frac{1}{m_i} \exp(\frac{5(GSI - 100)}{252}) \tag{10}$$

For a given GSI, σ_t/σ_c is a function of mi and can be graphically presented in the three dash lines in Figure 11 corresponding to a GSI value of 50, 60 and 70 respectively. Clearly, Hoek-Brown criterion gives an extremely low σ_t/σ_c ratio in comparison with that given by Mohr-Coulomb failure criterion. Therefore, for a rock mass where tensile strength dominates material failure, Hoek-Brown and Mohr-Coulombwould give substantially different results.



The ratio of rock mass tensile strength/UCS

Figure 11. Difference of tensile strength/UCS ratios involved in Hoek-Brown and Mohr-Coulomb criteria.

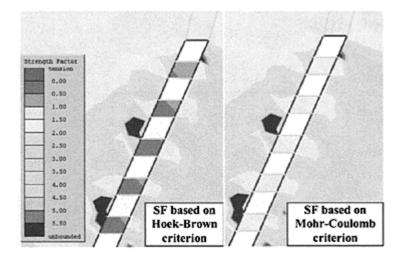


Figure 12. Different results from Hoek-Brown and Mohr-Coulomb criteria.

In demonstration, the stability of cemented backfill beam used in under hand cut-and-fill mining was analysed using Phase2 (RocScience, 2003). A six meter thick ore body is mined by multi level top-down mining at a level interval of 10 m. Of which 4 m is filled and 6 m is left open. Numerical modelling using Hoek-Brown criterion showed that the fill beams made of cemented aggregate fill at 5% cement would fail in tension while Mohr-Coulomb indicated stable fill beams could be achieved as shown in Figure 12. Which result was correct? With very successful practical application of this cut and fill mining method (P.Rocque, 2000), it appears that Mohr-Coulomb criterion is more appropriate for modelling material with relatively high tensile strength/UCS ratio such as cemented backfill.

7 ADVANCED MODELLING IN MIMICKING DEFORMATIONAL BEHAVIOUR OF INTACT ROCK AND ROCK MASS DISCONTINUITIES

In common mining geomechanics modelling, either Mohr-Coulomb or Hoek-Brown criterion is used for excavation stability examination. A limitation resulted

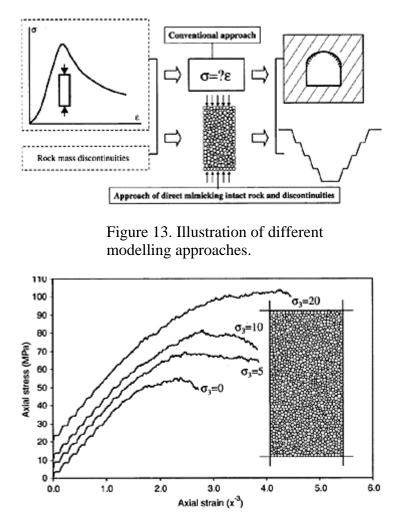


Figure 14. Stress-strain behaviour of the particle assembly in biaxial testing.

from applying these criteria is the inability in capturing the complicated post failure behaviour of rock mass. As a matter of fact, the majority of rock mass immediately surrounding an excavation in mining environment works at its post failure portion of full stress-strain curve. Numerical simulation of this part of rock mass behaviour for mining application purpose is still impractical.

A new advance in rock mechanics modelling has been to create a numerical material which contains bound particles with stress-strain behaviour that is in good agreement with that of the prototype intact rock exhibited in lab testing and structural features that match field geotechnical structural mapping and mechanical behaviour. As shown in Figure 13, with a rock mass that was simulated with behavioural characteristics that

match both the intact rock and rock mass discontinuities, the approaches of direct modelling rock mass properties avoid the assumptions that are made in conventional numerical computational approaches.

As an example, a heavily jointed rock slope was modeled using discrete element method package PFC2D (Itasca, 1999) in three steps (Wang and Tannant, 2003):

Step 1: create particle assembly that mimics intact rock behaviour as shown in Figure 14 and Figure 15.

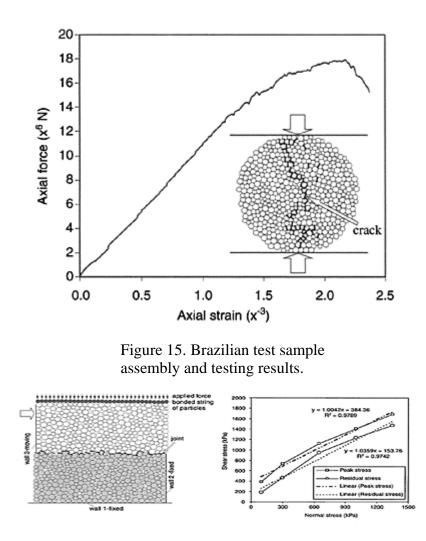


Figure 16. Direct shear modelling with PFC.

Step 2: create joints that mimics joint properties as demonstrated in Figure 16.

Step 3: create a particle assembly that mimics jointed rock mass.

Step 4: model the stability of a jointed rock slope.

The model created with particle assembly exhibiting the same or approximately same behaviour of both the intact rock and discontinuity of the prototype rock mass is reasonably believed to behave the same as reality. Figure 17 shows an example of slope failure occurred in a heavily jointed rock mass modelled with PFC2D.

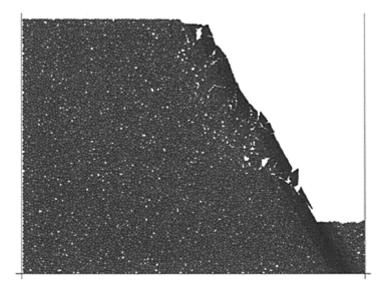


Figure 17. Failure of a heavily j ointed rock slope modelled with PFC2D.

8 CONCLUSION

Numerical stress modelling has been playing an increasingly significant role in mine planning and design for underground and surface mining. The quantitative results produced by numerical modelling give engineers an opportunity of understanding more scientifically the impact of mining induced stresses on mining excavations and finding out the optimal mining sequence and ground control strategies. However, the mechanical behaviour of complicated rock mass is hard to be fully captured by approximately half a dozen input parameters used in numerical computation, especially the non linear stress-strain behaviour of rock material incorporated with discontinuity. The nature of wide variation of rock mass properties and the configuration of mining excavations present a paramount importance of modelling calibration.

For the widely used elastic stress modelling, the most important parameter would be the rock mass strength on an engineering scale. This can be better determined by using back analysis where existing stress induced rock mass failures are observable. A conservative rock mass strength may also be found from the stable excavations in a mine site where new extensions at deeper levels are being studied.

In-situ rock mass stress is as important as rock mass characteristics. However, the importance of its orientation might be neglected in comparison with its magnitude at a mine site where no in-situ stress measurement is available. An incorrect assumption of in-situ stress orientation would easily produce opposite results which would be misleading and hard to notice.

For the commonly used rock mass failure criteria, the Hoek-Brown and Mohr-Coulomb criteria, an applicability exists due to the tremendous difference in the tensile strength/UCS ratios given by these two criteria. It may be appropriate to try Hoek-Brown criterion for brittle rock with low tensile strength while using Mohr-Coulomb for rock mass or geo-materials that have relatively high tensile strength.

Modelling a rock mass through modelling intact rock and rock mass discontinuities provides a good approach to capturing the non-linear deformational behaviour of rock mass while avoiding the assumption of rock mass failure criteria and complicated stressstrain constitution model.

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Axial force distribution of friction-anchored rockbolts

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ABSTRACT: In this paper, firstly the development of an axial force measurement system for a frictionanchored expanded-steel-tube-type rockbolt (EST rockbolt) is described. Its performance is verified with laboratory full-scale pull tests. The models to be used for the numerical analysis are discussed through the numerical simulations of the pull tests, which successfully trace the measured results especially in the axial force distribution behaviours. Secondly, the developed axial force measurement system for the EST rockbolt is applied to a field application. In-situ pull tests were performed at a construction site of Takadayama Tunnel where long facebolting with connectable EST rockbolts are used to secure the face stability. The EST facebolt for the axial force measurement was installed at a face and the axial force distributions were measured during excavation of the tunnel. A three-dimensional numerical analysis in which the construction procedures were fully considered was used to attempt to reproduce the measured axial force behaviours.

1 INTRODUCTION

It is important to know the generated axial force distribution along a rockbolt in evaluating the reinforcement effects of rockbolting or facebolting. Hence, the measurement of the axial forces of rockbolts has been routinely performed at some sections of each tunnel. Friction-anchored expanded-steel-tube-type rockbolts (referred to as EST rockbolts henceforth) have been used where the early-time function is required and/or the loss of grout due to water ingress is expected. In addition, they have been used for facebolts since excavation machines such as roadheaders can easily cut them. Recently, long facebolts composed of thread-connected EST rockbolts are also used (e.g.

Adachi & Ogawa, 2002). The EST rockbolt is originally developed by Atlas Copco. as Swellex bolt in 1977 (Stillborg, 1986).

The anchoring mechanism of an EST rockbolt causes difficulty in measuring the distributed axial forces acting along this type of rockbolts installed in-situ. Such a need motivated us to develop a multiple-point measurement system for axial forces in EST rockbolts.

As background to this study, some of the authors have studied the axial force distribution acting along long cablebolts. They have recognized the importance in that the numerical model of a rockbolt should take into account the mechanism of the shear deformation of grout and the slip at the bolt/grout or grout/boreholewall interface. They have carried out pull tests of long-embedded cablebolts and introduced methods to determine characteristic parameters of a model for a cablebolt (Aoki et al., 2002 & 2003). With the parameters determined, the numerical simulations of the pull tests well-traced the test results. They have also successfully applied this method to rockbolts and GFRP-tube facebolts (Tani et al., 2004 & Otsuka et al, 2004). The bolt model with the parameters determined from the in-situ pull tests can be used in the numerical analyses of tunnelling for quantitative evaluation of the reinforcing effects.

In this paper, firstly, the development of the axial force measurement system for the EST rockbolts is described. Secondly, laboratory pull tests with full-scale mockup boreholes are carried out in order to verify the function of the axial force measurement system and the similarity of behaviours of the EST rockbolts with and without the axial force measurement system. Based on the test results, the parameters for the rockbolt model are determined with the method proposed by the authors (Aoki et al., 2002). The numerical simulations of the pull tests are carried out to verify the reproducibility with the test results. An improved model is also proposed so as to better express the particular characteristic of the behaviour of the EST rockbolt.

The developed axial force measurement system is applied at a road tunnel in argillaceous soft rocks where long facebolting (20 m in length) with connected EST rockbolts are used. In order to determine the parameters of the rockbolt model installed in-situ, pull tests were carried out. A 16 m-long facebolt with the multiple-point axial force measurement system was installed at a tunnel face and the axial forces were measured during excavation. Furthermore, threedimensional numerical analyses on the tunnel excavation were performed and they successfully simulated the bolt axial force distribution.

2 DEVELOPMENT OF AXIAL FORCE MEASUREMENT SYSTEM

2.1 Construction

Strain gauges have been widely used to measure the strains of a re-bar which can be converted to the axial forces and/or bending moments. In this connection, the same technology has also been applied to the measurement of axial forces distributed along a rockbolt of a fully-grouted type. The measured axial force distributions are useful in understanding the extent of the loosened zones developed around an excavated underground opening so that one can examine the appropriate support system to be used. However, direct application of strain gauges on the surface of the EST bolt will result in problems as follows due to its installation mechanism:

- when the folded steel tube of the body of the EST rockbolt is expanded with hydraulic pressure and the surface is pressed on the borehole wall, the strain gauge will be crushed and damaged;
- in the expansion process, the steel of the tube is yielded with a large strain that results in the bond failure of adhesive agent of strain gauges.

To solve these problems, the structure illustrated in Figure 1 is worked out. The strain gauge is glued on the surface of the EST rockbolt so as to measure the strain in the axial direction; it is protected from crushing during the expansion with resin material filled in the space between the two thin steel tubes welded along the axial direction of the EST rockbolt; the two steel tubes also restrain the body steel of the EST rockbolt from generating too much large strain in the axial direction. Since only the strain at one side of the EST rockbolt can be measured with this measurement system, the measured strain will include the bending effect. However, as far as the EST rockbolt is confined in the borehole, the measured strain is attributed to the axial strain and would result in sufficient

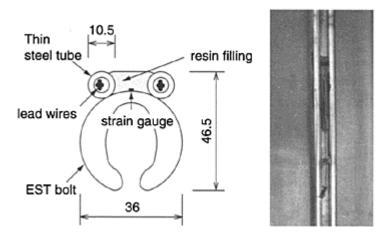


Figure 1. EST rockbolt instrumented for axial force measurement.

Table 1. Specification of EST rockbolt—Super Swellex.

	Average tensile strength	Cross section area [mm ²]		
Material	[kN]	Plain	Instrumented	
SS1232	220	481	539	

approximation. The EST rockbolts used in this study are Super Swellex of Atlas Copco, of which specification is given in Table 1.

Laboratory pull tests of the bolt material are conducted. The test specimens are prepared as follows: firstly, each of the EST rockbolts is expanded in a 50 mm-diameter steel form and is taken out; it is cut at each end where a flange is welded with its plane perpendicular to the rockbolt axis; a strain gauge is installed at the corresponding position to that illustrated in Figure 1; similarly, the test specimen of a plain rockbolt is prepared. During the pull test with a laboratory loading machine, the rotation at each end of the specimen is fixed.

Figure 2 shows the applied axial stress-measured axial strain relationship of the bolt material of the EST rockbolt. The axial stress is simply derived from the applied pull load divided by the cross-sectional area of the bolt material. The curve for the plain bolt showed significant nonlinearity in higher levels of the axial stress. This would be due to the distorted hollow structure with the non-axisymmetrical cross section resulting in partial stress concentration.

On the contrary, the stress-strain curve of the bolt with the axial force measurement system is rather linear in comparison with that of the plain bolt. However, the strain measured is smaller than that of theoretical prediction depicted with the dashed line. This would result from disproportionate deformation due to the restriction by the thin steel tubes. Although such differences in the stress-strain curves of the two

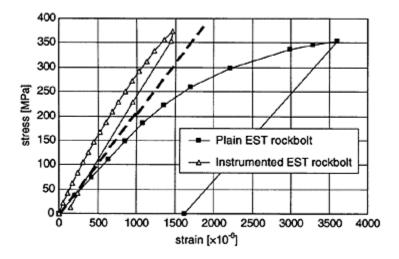


Figure 2. Axial stress-strain curves obtained from laboratory pull tests of bolt materials of EST rockbolts.

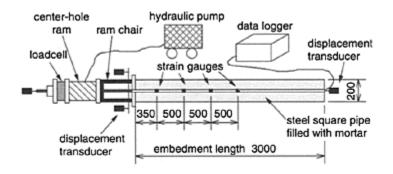


Figure 3. Layout of full-scale pull tests.

types of the rockbolt are observed, they are predicted to become close under the confined condition in a borehole where bending is restricted.

2.2 Pull test of rockbolts

Full-scale pull tests of the EST rockbolts are performed to verify the function of the axial force measurement system and the similarity of behaviours of the EST rockbolts with and without the axial force measurement system.

The layout of the pull test is shown in Figure 3. The mockup borehole is made by a 3 m-long square steel pipe filled with mortar. The borehole is reserved with an inner form made of a PVC tube. The borehole diameter is selected as 48 mm for plain EST rockbolt and 54 mm for that with the axial force measurement system. The properties of mortar on the test day are given in Table 2. Each of the 4 m-long EST rockbolts with and without the axial force measurement system is installed with high water pressure of 30 MPa and then, the rear protruded part of 1 m long is cut out. The embedment length is 3 m.

In each pull test, displacement transducers are installed at the basal location of the installed EST

UCS* [MN/m ²]	Young's modulus [GN/m ²]	Poisson's ratio
23.1	18.8	0.24

*: At 7 days of material age.



Figure 4. Crosscut of the tested borehole.

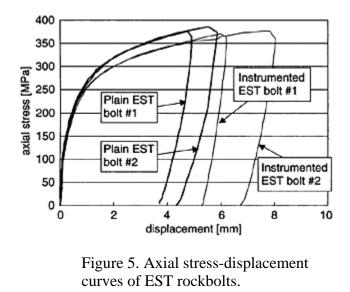
rockbolt so as to measure the displacement with minimum influence of movement of the free part of the rockbolt. For the EST rockbolts with the axial force measurement system, strain gauges are installed at the positions depicted in Figure 3 and the strain changes are measured during the pull test. The measured strains are converted with the Young's modulus of steel and the cross-sectional area.

Figure 4 shows the crosscut of the test borehole installed with the EST rockbolt with axial force measurement system after the pull test.

In testing, firstly, initial load of 5 kN is applied so as to remove the slacks of assembled test devices. Subsequently, the pull load is increased in 5 kN steps up to 200 kN and then unloaded.

The results of the pull tests in the form of applied axial stress-displacement curves are shown in Figure 5. Since the cross-sectional areas of the EST rockbolts with and without axial force measurement system are different, the applied axial stresses converted from the pull load are used here for appropriate comparison. It is found that the two tests conducted for each type of rockbolts are quite reproducible.

The stress-strain curves of the EST rockbolts with axial force measurement system correspond well with those of the plain EST rockbolts up to the applied axial stress level of about 250 MPa (120 kN in load) and subsequently, they slightly differ at higher stress levels. This can be explained by considering the particular configuration at the end of the EST rockbolt as illustrated in Figure 6. Since the EST rockbolt



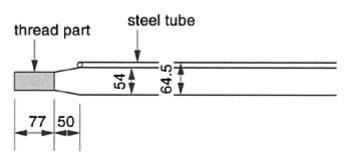


Figure 6. End configuration of EST rockbolt with axial force measurement system.

has curved shape in approximately 50 mm part from the threaded end coupling, the thin steel tubes for protection are not fixed in this part. Hence, the actual stress acting in this part is at a higher level, which results in great nonlinear and large strain. From these considerations, we can consider that the behaviours of the applied axial stress-displacement relations of the EST rockbolts with and without axial force measurement system are reasonably correspondent.

The distributions of the axial forces measured at the three different pull load levels (e.g. for the EST rockbolt No.2) are drawn in Figure 7. Here, the axial forces are simply derived from the measured strains multiplied by the cross-sectional area and the Young's modulus. As found for the other types of rockbolts and cablebolts (Aoki et al., 2002 & 2003, Tani et al., 2004 & Otsuka et al., 2004), quite linear and parallel distributions of the axial forces are observed.

However, it is different in that the peak-out distribution of the axial forces near the borehole collar is recognized where the pull load directly propagates to the deeper position (by approximately 35 cm from the borehole collar) of the EST rockbolt. Since the EST rockbolt is hollow and fixed by expansion, it presumably shrinks in diameter at high stress levels resulting in the loss of the friction at the bolt/ borehole-wall interface.

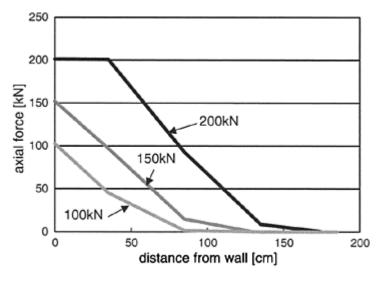


Figure 7. Axial force distribution of EST rockbolt No.2.

2.3 Numerical simulation of pull tests

2.3.1 Model for rockbolts

In the model for rockbolts, the shear resistance and/or bending resistance at discontinuities are often taken into account. However, such resistances for the EST rockbolt seem to be relatively small by comparison with the fully-grouted rebar-type rockbolts, except for such resistances resulting from contribution of the generated axial forces. On the other hand, the behaviours of shear deformation and slip at the bolt/borehole-wall interface need to be taken into consideration in the model for the EST rockbolt since they are the major characteristics for friction-anchored rockbolts.

A model for fully-grouted cablebolts proposed by Brady & Lorig (1988) as illustrated in Figure 8 can express such shear behaviours. The grout shear deformation is expressed by shear springs with their stiffness of *kbond* and grout slip is expressed by sliders with their cohesive strength of *sbond*. The authors have proposed the methods to quantitatively determine those parameters for the cablebolt model based on their results of pull tests of long-embedded cablebolts and have found that the numerical simulations successfUlly trace not only the pull load-displacement curves but also the internal axial force distribution (Aoki et al., 2002). Here, the same methods are applied to the EST rockbolt; *kbond* would represent the local shear deformation of rock at the borehole wall; and *sbond* would represent the frictional resistance at the bolt/borehole-wall interface.

2.3.2 Determination of model parameters

(1) Constant sbond model

The method to determine *kbond* is illustrated in Figure 9. The initial gradient (P/ξ_0) of pull load (P)—displacement (ξ) curve obtained from the pull

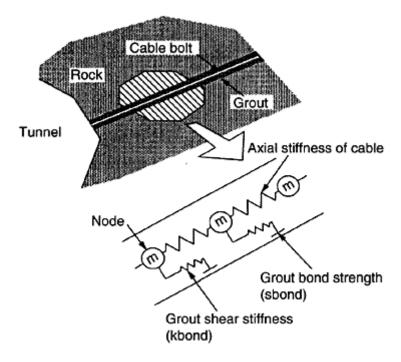
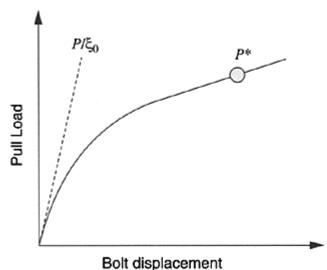


Figure 8. Model for rockbolts (after Brady & Lorig, 1988).



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Figure 9. Method to determine *kbond*.

test of the rockbolt is used in Equation 1 where A_b and E_b is the cross-sectional area and Young's modulus of the rockbolt, respectively.

$$kbond = \frac{1}{A_b E_b} \left(\frac{P}{\xi_0}\right)^2 \tag{1}$$

It was originally derived by Saito & Amano (1984) for a model of a fully-grouted rebartype rockbolt based on the consideration of axial and shear stress balance at the infinitesimal segment of a rockbolt. Here, the cross-sectional area of the substantial part of the hollow EST rockbolt is used for A_b . *kbond* for the EST rockbolt No.2 is determined as 2.67×10^9 N/m².

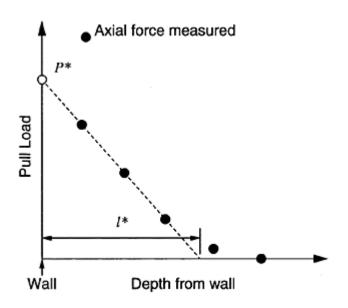


Figure 10. Method to determine sbond.

The method to determine *sbond* is illustrated in Figure 10 in which the axial load distribution along the rockbolt embedded in rock is drawn. It has been found that the distribution tends to have a linear part and hence, the gradient can be representative of the shear resistance (Aoki et al., 2003). Thus, *sbond* can be derived from Equation 2.

$$sbond = \frac{P}{l^*} \tag{2}$$

The axial force distribution obtained from the pull test is shown in Figure 7. *sbond* determined from the gradient of the linear part of the axial force distribution of the instrumented EST rockbolt No.2 at the pull load level of 200 kN by means of the linear regression technique is 1.64×10^5 N/m.

(2) Decaying sbond model

It is preferable that the phenomenon that the EST rockbolt loses its friction with the borehole wall at high stress levels, as observed in Figure 7, be expressed in the rockbolt model. This can be achieved by varying the quantity of *sbond* in the rockbolt model. As shown in Figure 11 by a solid line, *sbond* is constant while the shear displacement between the rockbolt and borehole wall is small; and then, *sbond* is set to decay with the shear displacement from a certain displacement A, and to reach zero at and after another certain displacement B. This is referred to hereafter as the decaying *sbond* model.

Displacement A can be approximately determined as the rockbolt displacement near the borehole collar at the pull load level of 120 kN (i.e. 250 MPa) where the displacement starts rapidly increasing as shown in Figure 5. This results in 0.68 mm. Displacement B can be presumed as the displacement at the pull load level of 150 kN where the friction loss is yet to be observed as shown in Figures 5 and 7. This results in 1.5 mm.

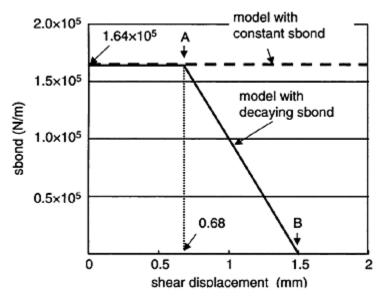


Figure 11. *sbond*. variable with shear displacement.

2.3.3 Results of simulation

The numerical simulations of the pull tests are performed to verify how appropriate the rockbolt model behaves. A finite difference analysis code, FLAC (Itasca, 2000) is used for the numerical analyses for the simulations.

(1) Constant sbond model

The pull load—displacement curve of the numerical simulation with the rockbolt model with constant *sbond* is shown in Figure 12a with that of the test results of the instrumented EST rockbolt No.2 for comparison. The curve of the numerical simulation significantly differs from that of the test result at higher pull load levels.

The traces of the axial forces at each measured location with the applied pull load are drawn in Figure 12b. As the pull load increases, the axial force measured at shallower locations starts increasing; subsequently, that of the next shallower locations starts increasing. These behaviours are on the whole well reproduced by the numerical simulation, especially in the region where the axial force of each location is relatively small. However, they still significantly differ in the higher axial force levels. Incidentally, the data in the pull loads levels between 60 and 95 kN are missing due to noise.

(2) Decaying *sbond* model

The numerical simulation of the pull test with decaying *sbond* model of the EST rockbolt as given in Figure 11 is performed. The results of the simulation and test are compared in Figure 13.

The pull load-displacement curve of the simulation is still different from that of the test result although the peak-out behaviours are somewhat alike, as shown in Figure 13a. This is presumably due to the nonlinear behaviour of the deformation of the bolt

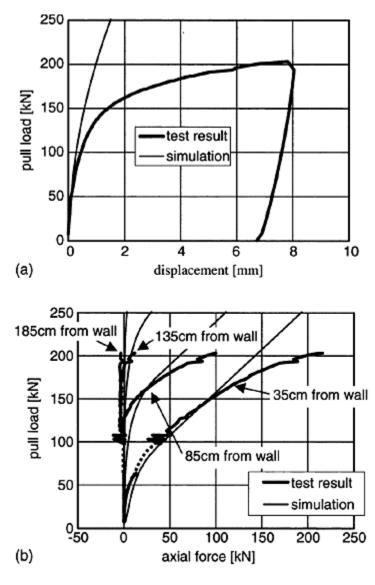


Figure 12. Comparison of simulation and test results. (*sbond* constant model) (a) Pull load—displacement relationship. (b) Pull load-axial force relationshop.

material at the part of end section of the EST rockbolts, as described in Section 2.2 with Figure 6, which is not taken into consideration in the numerical model for the rockbolt.

For the traces of the axial forces of each measured location with the applied pull load as depicted in Figure 13b, the results of the numerical simulation can better explain the test results for the axial force of each location of the measurement points, even in the high axial force levels. This can be seen from a different point of view of the axial force distribution as shown in Figure 13c. The behaviour that the pull load propagates to a deeper location due to the loss of friction (or decay of *sbond*) is well expressed by the simulation.

2.3.4 Discussion

It was found that the EST rockbolt has nonlinear characteristics not only in the frictional behaviour but also in the load-displacement relationship of the bolt material. The nonlinearity in the frictional behaviour can be modelled by considering the decaying of the frictional resistance, *sbond*. With this model, the pull tests of the EST rockbolt are well traced by the numerical simulation in the axial force distribution behaviours. The differences in the pull load-displacement relations will be less significant in the analyses on the reinforcement effects by rockbolting in tunnelling since such behaviours are limited in the region around the rockbolt heads.

It is emphasised that the simpler numerical model with constant *sbond* may be still useful for the EST rockbolt in that the axial force distributions in the pull load levels up to 150 kN are reasonably well simulated by the numerical analysis and only those of very high axial force levels may not be traced by the model. This can be improved by adjusting the quantity of *sbond*. Such an approach would be of practical use in the engineering cases.

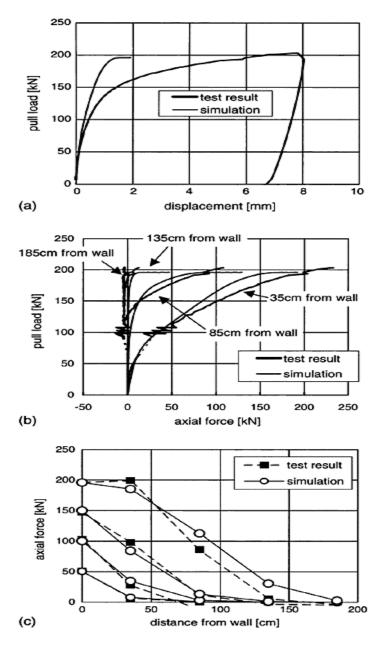


Figure 13. Comparison of simulation and test results. (Decaying *sbond* model) (a) Pull load-displacement relationship; (b) Pull load-axial force relationship; (c) Axial

3 FIELD APPLICATION

3.1 Outline of a tunnel construction project

The axial force measurement system for the EST rockbolt developed is applied at a tunnel construction project. Takadayama tunnel is a 1.7 km-long two-lane road tunnel for Kisei line of Kinki highway and located in the southwest region of Kii peninsula, which is south of Osaka. The rock comprises a melange of Paleogene shales in an accretionary prism.

While the bench excavation method was used, the top-heading face tended to collapse even though shotcreting was applied to the face. Therefore, long facebolting with connectable EST rockbolts (mainly 20 m in length) was applied to secure the face, as shown in Figure 14. The EST facebolts were very effective in improving the face stability while they were easily cut during excavation by a roadheader.

3.2 In-situ pull test of rockbolts

In order to obtain the parameters of the rockbolt model to be used in the numerical analyses of the reinforcing effect by the facebolting, in-situ pull tests of the EST rockbolt were carried out.

The layout of the in-situ pull tests is shown in Figure 15. The tests are conducted at a sidewall of bench. The EST rockbolts with the axial force measurement system developed are used. They are 4 m in length and of Super Swellex with specification shown in Table 1. Each of them are installed in a 54 mm diameter borehole made with a drill jumbo. The 30 cm section from the borehole collar is enlarged with 65 mm diameter drill bit and the rockbolt in this section is covered with 60.5 mm diameter steel pipe so as to avoid fixing the rockbolt in the shotcrete part (20 cm and more in thickness). The test procedures are similar to those described in Section 2.2; so that the detail is not given here.

The results of the pull tests in the form of pull load-displacement curves are given in Figure 16. Two tests were carried out and the repeatability is quite good. The axial force distributions at four pull load levels are drawn in Figure 17 for the EST rockbolt No.1. Up to 30 cm from the borehole collar is free from friction with the borehole wall, so that the pull load is directly transferred to that depth. Similar to the results of the laboratory large-scale pull tests, the behaviour of friction loss of the EST rockbolt at high pull load levels is observed.

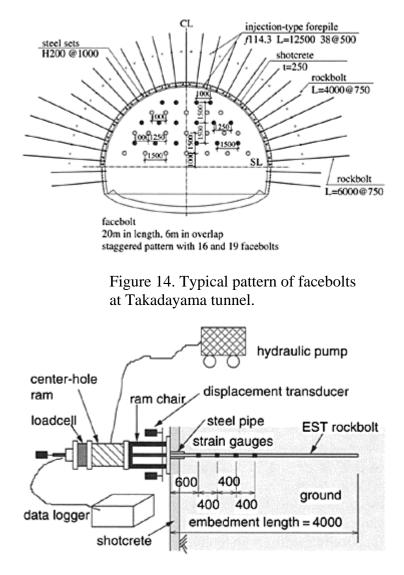
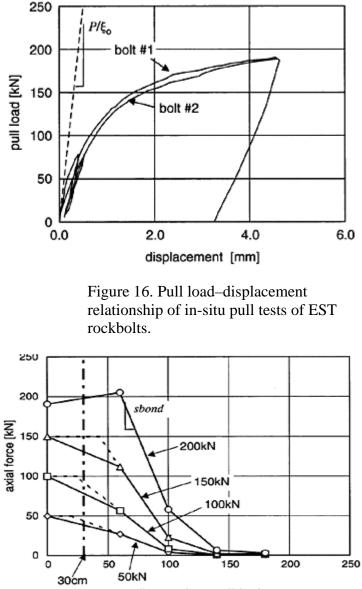


Figure 15. Layout of in-situ pull tests of EST rockbolts.

The parameters for the rockbolts are determined from the test results based on the method described in Section 2.3.2. *kbond* of 2.26×10^9 N/m² and *sbond* of 3.68×10^5 N/m are obtained where the maximum gradient of the axial force distribution at the pull load level of 200 kN is used to determine *sbond*.

The parameters for the decaying *sbond* model are also determined as shown in Figure 18 in the same manner as described in Section 2.3.2(2). Displacement A is determined as 0.2 mm corresponding to the pull load of 25 kN where the pull load-displacement curve departs from the initial linear part. Displacement B is determined as 0.9 mm

corresponding to the pull load of 120 kN where the region of friction loss from the borehole collar starts elongated from the initial friction-cut length of 30 cm.



distance from wall (cm)

Figure 17. Axial force distribution of in-situ pull tests of EST rockbolts (No. 1).

The results of the numerical simulation of the pull tests are compared with those of the tests in the pull load-axial force curves in Figure 19. The simulation results with the constant *sbond* model can satisfactorily follow the test results up to the pull load level of about 150 kN. Those with the decaying *sbond* model can trace the curves of the test results up to further high pull load levels.

3.3 In-situ measurement of axial force distribution of a facebolt

The EST facebolt with the axial force measurement system developed is installed in a face of Takadayama tunnel. At this section, the pattern facebolting is not executed and a single long facebolt of 16 m in length

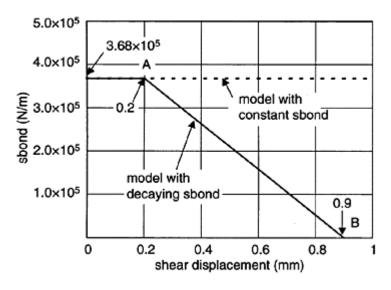


Figure 18. *sbond* for the simulation of in-situ pull test.

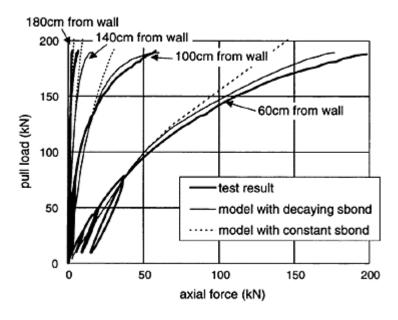


Figure 19. Pull load—axial force relationship of in-situ pull tests.

is installed for the technology development purposes. Such a long facebolt is assembled with four connectable EST rockbolts, 4 m long each.

The borehole is drilled from the face at a position 1 m above the top-heading floor and of 1.6 m left of the tunnel centre. The ground plan of the installation of the EST facebolt for measurement is shown in Figure 20. The eight measurement points, each in which a strain gauge is installed, are allocated to the mid part of the 16 m-long EST facebolt spaced by every 1 m. This is based on the findings of the preliminary three-dimensional analysis that the axial forces distribute with their peak located several metres behind the excavated face. A cylinder-shaped data logger of 8 channels is installed at the head of the EST facebolt (i.e. at the bottom of the borehole) and the data are stored in the memory every 30 minutes. Once the face reaches close to the data logger, it is retrieved with careful digging.

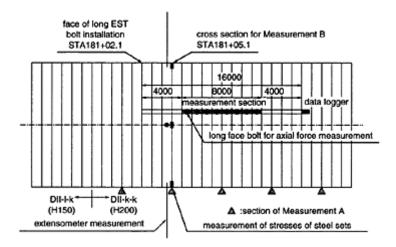
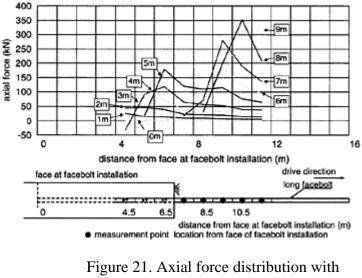


Figure 20. Ground plan of the installation of EST facebolt for axial force measurement.



advancing face.

The measured axial force distribution with the advancing face is plotted in Figure 21. In this graph, 0 m in abscissa means the location of the face where the EST facebolt is installed; and the number in each square represents the corresponding face location. For example, when the face is advanced by 4 m by every 1 m round, the axial force distribution has the peak at 6.5 m distance; that is 2.5 m ahead of the face at that time. On the whole, the peak of the axial force distribution is 2 to 3 m ahead of the face.

The maximum axial load measured is more than 300 kN but it is a false quantity since the bolt material should actually yield at around 220 kN. This results from the straightforward conversion to the axial force from the measured strain. It is worth pointing out that the axial force level of the facebolt is quite high in response to the extruding movement of the face.

In the region from 7 to 8 m from the installation face, the axial forces are low. The face observation record provided the situation that a very weak layer that could be easily dug by hand appeared there. The low axial forces would be due to the little friction to fix the facebolt in that region.

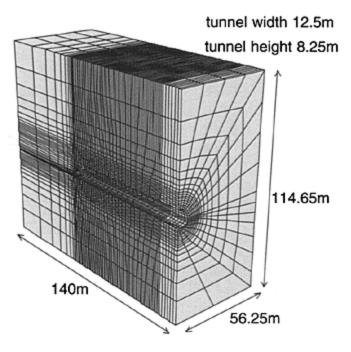


Figure 22. Ground model for the threedimensional analysis.

The results of the other measurements simultaneously conducted at the nearby sections showed more than 80 mm of convergence and more than 50 mm of settlement at the base of the top-heading support. The results of the extensometer measurement indicated the loosened region extended up to 6 m from the tunnel wall. The axial stresses acting on the steel sets are measured as exceeding the yielding stress of 240 MN/mm².

3.4 Three-dimensional numerical analysis on the axial force distribution behaviours

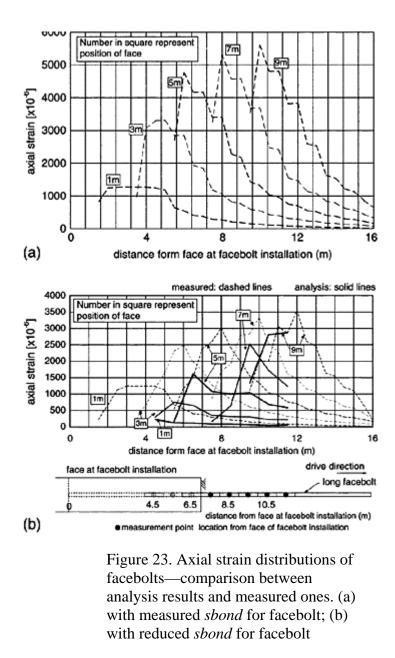
Using a finite difference analysis code, FLAC3D (Itasca, 2002), the tunnel excavation is simulated with a facebolt installed from an appropriate face. Figure 22 shows the ground

model for the numerical analysis. The half-section is analysed and the boundaries are fixed with rollers, except for the top plane being set free as the ground surface. The overburden from the tunnel crown is 55 m. The properties used for the numerical analysis are given in Table 3. The rock is modelled as an elasto-perfectly plastic body with Mohr-Coulomb yield criterion and the elastic and strength properties set so that the analysis results are in accordance with the convergence and other measured results following several trial-and-error runs.

The shotcrete is modelled with shell elements and the steel sets are modelled with connected beam elements. The excavation and installation of the supports are reproduced so as to be the same as the actual construction procedures. The radial pattern rockbolts are not considered in the analysis. The rockbolt model with constant *shond* is used for the facebolt.

Material	Parameter	Unit	Quantity
Rock	Young's modulus	MPa	500
	Poisson's ratio		0.35
	Density	kg/m ³	2300
	Cohesion	MPa	0.05
	Angle of int'l friction	degree	30
Facebolt for measurement	kbond	N/m ²	2.26×10 ⁹
	sbond	N/m	1.00×10^{5}
Shotcrete	Thickness	m	0.25
	Young's modulus	GPa	4.0
	Poisson's ratio		0.2
Steel sets	Cross sectional area	m^2	6.35×10 ⁻³
	Moment on inertia	m^4	4.72×10 ⁻⁵

Table 3. Properties used for the numerical analysis.



The results of the numerical analysis are compared with the measured ones in the axial strain distribution of the facebolt, as shown in Figure 23. This is because part of the facebolt is in a yielded state and stresses cannot be compared in the state exceeding the yield strength of the facebolt.

The numerical analysis with the *sbond* quantity determined from the in-situ pull tests resulted in the axial strains distributing in very large levels, which are far different from

the measured axial strain distribution, as given in Figure 23 a. Although the nonlinear behaviour of the facebolt at high stress levels is not taken into account, such difference did not seem to be compensated if such a model is used. Hence, it is assumed that the rocks at the facebolt installation are in poorer condition than those at the location where the in-situ pull tests were conducted. Based on this consideration, *sbond* for the facebolt was decreased down to 30% of the quantity determined from the in-situ pull tests.

The results of the analysis with the reduced *sbond* are shown in Figure 23b. When the face is advanced by 7 and 9 m from that of the facebolt installation, the axial strain distributions obtained from the numerical analysis correspond well with those of the measured axial strains not only in their quantities but also in the distribution shape. The phenomenon of the axial strain distribution having its peak at the location about 3 m ahead of the face is in good accordance. On the contrary, the measured axial strains of the facebolt is smaller than those of the analysis results with the face advance up to 5 m. This would result from the influence of the existence of the very weak layer of rocks, as described in Section 3.3.

4 CONCLUDING REMARKS

In this paper, firstly, the development of the axial force measurement system for the EST rockbolt is described through laboratory pull test. The numerical models for the EST rockbolts are investigated. It is found that the methods to determine the parameters of the rockbolt model from the results of a pull test are also useful for the EST rockbolt. Through the investigation with the field measurements, a series of technology from the measurement to the evaluation of the reinforcement effects by means of facebolting are constructed.

The authors plan to proceed with their study by performing three-dimensional analyses of the tunnel excavation for the sections where the pattern facebolting is executed. The reinforcing effects by facebolting can be assessed by comparison of numerical analyses of the tunnel excavations with and without facebolting. Cases of the use of different types of facebolts may be investigated as well.

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A case study on stochastic fracture geometry modeling in 3-D including validations for a tunneling site in USA

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ABSTRACT: Eight hundred and fifty nine fractures of a gneissic rock mass were mapped using 16 scanlines placed on steep rock exposures that were within 300 m of a tunnel alignment before the tunnel excavation. These data were analyzed using the software package FRACNTWK to find the number of fracture sets that exist in the rock mass, 3-D fracture frequency for each set and the probability distributions of orientation, trace length, fracture size in three-dimensions (3-D) and spacing for each of the fracture sets. In obtaining these distributions corrections were applied for sampling biases associated with orientation, trace length, size and spacing. Developed stochastic 3-D fracture network for the rock mass was validated by comparing statistical properties of observed fracture traces on the scanlines with the predicted fracture traces on similar scanlines. The one-dimensional (1-D) fracture frequency of the rock mass in all directions in 3-D was calculated and is presented in terms of a stereographic plot. The 1-D fracture frequency prediction made along the tunnel alignment direction was found to be in excellent agreement with the observed values obtained about one year later during the tunnel excavation.

1 INTRODUCTION AND GEOLOGICSETTING OF THE SITE

The Metropolitan Water District of Southern California (MWD) began construction of its 70 km (43.8 miles) Inland Feeder in April 1997 for which the estimated cost is \$1.2 billion. The Inland Feeder will convey as much as 2.46 billion liters (650 million gallons) of water a day from the Devil Canyon Afterbay of the California Aqueduct of the State

Water Project in San Bernardino County to the Colorado River Aqueduct (CRA) and the recently completed Diamond Valley lake, in Riverside County. The project consists of constructing approximately 41.6 km (26 miles) of pipelines and 28.8 km (18 miles) of tunnels. Construction will last approximately eight years.

The first segment, known as the San Bernardino Mountains Segment of the Inland Feeder, consists of two tunnels with a combined length of 15.7 km (9.8 miles) through the San Bernardino Mountains. These are known as the Arrowhead West and Arrowhead East Tunnels. The Arrowhead East Tunnel site is considered in the paper. For further information about the Inland Feeder project, the reader is referred to Gallanes et al. (1996).

The Arrowhead East tunnel is located in the southern San Bernardino Mountains, a relatively young mountain range uplifted in Quaternary time along and north of the San Andreas fault system. The mountain range, which is part of California's east-west trending Transverse Ranges Province, is about 96.0 km (60 miles) long and 32.0 km (20 miles) wide, and bounded on the north by the Mojave Desert and on the south by the San Bernardino Valley.

The mountain range is underlain primarily by crystalline igneous and metamorphic rocks of Precambrian to Mesozoic age. The metamorphic rocks consist mostly of mafic and quartzo-feldspathic gneiss with schist and calcareous to dolomitic marble interbeds. The igneous rocks include quartz monzonite, granodiorite, and quartz diorite. These metamorphic and granitic rocks account for about 30 and 67 percent, respectively, of the rock anticipated at tunnel depth along the alignment. Conglomeratic sandstone accounts for the remaining 3 percent.

Several major faults cross the area. The largest and most significant fault is the northwest-trending San Andreas fault system, which defines the southern boundary of the San Bernardino Mountains. The San Andreas fault system in the project area consists of two separate strands; the South Branch and the North Branch. Both faults are considered active for design of the project. The South Branch is the larger and more continuous of the two and trends sub-parallel to the tunnel alignment approximately 610–1219 m (2,000–4,000 feet) to the south. The North Branch also trends sub-parallel to the alignment and crosses the tunnel near the eastern portal. Several other North-East to East-West trending faults cross the tunnel and are considered active or potentially active subsidiaries of the San Andreas fault system.

Numerous other large-scale discontinuities including faults, shears and lineaments cross the project area. The orientation of these features generally correlates with patterns associated with the general regional north-south principal stress. Fracture orientations within the rock mass also reflect the influence of the regional stresses and the San Andreas fault system. Fracturing nearer the San Andreas fault system tend to be more pervasive with large zones of brecciated rock. Fracture characteristics for this study were derived from exposures that were about 1830 m (6000 feet) away from the intensely fracture zones near the San Andreas fault.

The degree of difficulty of tunnel excavation, time required for tunnel excavation, the associated cost of tunneling, the tunnel stability, tunnel rock mass support system and water inflows into the tunnel depend on the fracture network of the rock mass around the tunnel. This paper focuses on the fracture characterization performed in the gneissic rock mass of the Arrowhead East Tunnel site.

Kulatilake (1998) completed a software package named FRACNTWK based on information given in more than ten journal papers that he and his co-workers published between 1984 and 1997 on the topic of fracture characterization and network modeling (Kulatilake and Wu 1984a, b and c, Kulatilake 1985, Kulatilake et al. 1990a, 1990b, 1993a, 1993b, 1996 and 1997, Wathugala et al. 1990). This package can be used to analyze discontinuity data obtained from boreholes, rock cores, scanlines and 2-D exposures such as rock outcrops, tunnel walls, tunnel roofs, etc. to perform fracture characterization and network modeling for discontinuous rock masses and to generate rock discontinuity systems in 3-D rock masses. Some computer programs from this package were used: (a) to determine the number of fracture sets and their orientation distributions; (b) to study the effect of orientation sampling bias on the orientation distribution of fracture sets; (c) to estimate the spacing distribution along the mean normal vector direction of each of the fracture sets; (d) to predict the 1-D fracture frequency in any direction in 3-D; (e) to predict the 1-D fracture frequency along the tunnel axis direction; (f) to estimate the trace length distribution and fracture size distribution in 3-D for each fracture set correcting for sampling biases; (g) to estimate the 3-D fracture frequency parameters such as the block size, number of blocks per unit volume and number of fractures per unit volume, and (h) to build a fracture network system in 3-D including a validation. The obtained results are given in this paper including appropriate discussions and conclusions.

2 FRACTURE MAPPING

The fracture data for the gneissic rock mass were collected from scanline surveys conducted on inclined surface outcrops. The terrain was very steep and rugged. Therefore it was not possible to conduct fracture mapping through the area sampling technique even though the area sampling technique is better than the scanline sampling technique in collecting fracture geometry data. The rock exposures that were within about 300 m from the tunnel alignment and showed existence of fracture sets and accessibility to conduct fracture mapping were selected to perform scanline fracture mapping. Twelve horizontal or sub-horizontal and four almost vertical scanlines drawn on sub-vertical exposures having dip angle between 60° and 90° were used to conduct fracture mapping in the gneissic rock mass to obtain the fracture data. The locations of the 16 scanlines (LS 7-LS 22) with respect to the Arrowhead East Tunnel alignment are shown in Figure 1. Note that the scanline fracture mapping was conducted before excavating the tunnel. None of these 16 scanlines fall along the tunnel alignment. A total of 859 fractures were mapped from these scanlines. This set of fractures provided a good representative sample for the tunnel site. The fracture traces appearing on rock exposures indicated that the rock mass has about 3 sub-vertical fracture sets and one sub-horizontal fracture set. Twelve horizontal or sub-horizontal scanlines were used to sample the sub-vertical fracture sets. To sample the sub-horizontal fracture set, four almost vertical scanlines were used. Figure 2 shows a typical rock exposure and a scanline used for fracture mapping. The existing irregular outer-boundaries on the outcrops were used in providing information about termination of fracture traces that intersected the scanlines. The dip direction and dip of the outcrop, the global location, trend and plunge of the scanline, the rock type and

the exposure condition of the rock mass (unweathered, weathered or altered) were recorded. For fractures that intersected the scanline, the intersection distance, strike, dip, apparent dip, semi trace length on each side of the scanline, termination type for each trace and aperture were recorded. Fracture traces less than 0.5 m were not recorded. All of the fracture data collected in the field were entered on a scanline survey logging form.

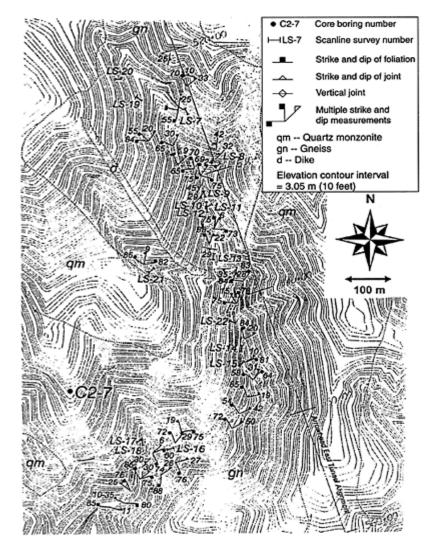


Figure 1. Relative locations of scanlines LS 7 through LS 22, borehole C2–7 and rock types with

respect to the alignment of Arrowhead East Tunnel.



Figure 2. A typical rock exposure used to conduct a scanline survey.

3 ORIENTATION DISTRIBUTION MODELING

The fracture data were analyzed according to the clustering algorithm of Shanley and Mahtab (1976) and Mahtab and Yegulalp (1984) to find the dense points and the resulting fracture sets. Even though this methodology suggests a procedure for finding the optimum number of fracture sets using three objective functions, for the analyzed data, it was not possible to find a unique value for the optimum number of fracture sets only from the results of this procedure. A number of fracture sets between three and five was found to be suitable according to the results obtained from the applied method. Therefore, the quality of the separation between the fracture sets on the equal area plot was considered, in addition to the observations made in the field on the apparent orientation of fracture traces appearing on inclined rock outcrops in the region the fracture mapping was conducted to make a decision on the optimum number of fracture sets. Fracture set delineation results obtained for the lumped fracture data coming from all the line surveys LS 7 through LS 22 are shown in Figure 3. These lumped fracture data indicate that 3 joint sets and 1 foliation set exist in the gneissic rock mass of the Arrowhead East Tunnel site. Because these four fracture sets were determined from the data obtained from all the scanlines selected in the region, they can be regarded as the fracture sets of the studied region (henceforth termed as regional fracture sets in the paper). This case study clearly showed that joint set delineation is not a trivial exercise and it requires good geologic and

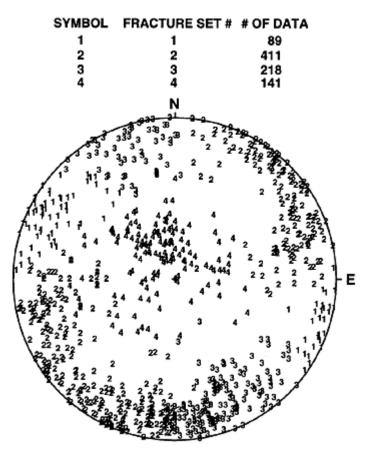


Figure 3. Delineated fracture sets on a upper hemispherical polar plot for the combined fracture data from scanlines LS 7 through LS 22.

engineering judgment in order to arrive at realistic solutions. Out of the obtained four fracture sets, fracture set number 2 shows the highest orientation variability (see Fig. 3).

Table 1. Number of fracture sets obtained from each scanline and goodness-of-fit results of Bingham distribution for orientation data.

Scanline number Nobs.	Fracture set	Npts.	Mu3	Chi-sq. test
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				Trend (°)	Plunge (°)	Р
LS 7	78	1	9	80.29	15.02	DISPT
		2	50	166.68	6.66	0.025
		3	19	255.08	64.94	DISPT
LS 8	34	1	34	77.99	30.00	DISPT
LS 9	44	1	34	328.57	20.32	DISPT
LS 10	36	1	25	45.40	8.33	DISPT
		2	11	337.71	28.27	DISPT
LS 11	16		16	54.00	4.75	DISPT
LS 12	70		16	55.21	12.28	DISPT
			40	289.37	9.30	>0.25
LS 13	134		115	197.78	10.19	< 0.01
LS 14	72		56	208.68	8.03	< 0.005
			11	339.88	62.41	DISPT
LS 15	27		27	47.88	6.40	DISPT
LS 16	61		61	258.08	18.88	0.005
LS 17	31		13	146.36	28.31	DISPT
		2	18	329.69	70.69	DISPT
LS 18	68	1	49	158.16	21.38	0.005
		2	19	343.32	63.87	DISPT
LS 19	66	1	52	160.37	10.49	>0.25
		2	14	297.36	51.56	DISPT
LS 20	20	1	20	77.08	67.36	DISPT
LS 21	25	1	25	308.31	85.74	DISPT
LS 22	77	1	14	155.56	3.80	DISPT
		2	24	236.88	0.78	DISPT
		3	23	288.17	1.84	DISPT
		4	16	342.42	68.29	DISPT
LS 7–22 Fol.	141	1	141	326.92	75.63	< 0.005
LS 7-22 Jnt.	718	1	89	294.27	7.79	DISPT
		2	411	224.52	4.97	< 0.005
		3	218	159.59	9.35	< 0.005

Nobs.=Number of fracture traces observed on the scanline.

Npts.=Number of fracture traces belonging to the fracture set.

Mu3=Upward mean normal vector direction for fracture set orientation.

Chi-sq. test=Chi-square test; Fol.=foliation; Jnt.=joint.

P=Maximum significance level at which Bingham distribution can be used to represent the

statistical distribution of orientation (a minimum of 0.05 is required to represent the orientation data by a Bingham distribution).

DISPT=Data were insufficient to perform the test.

Eight hundred and thirty one fracture data were also available from a vertical borehole (C2-7 in Fig. 1) of length 396 m. Intended tunnel depth was about 25 m above the bottom level of borehole C2–7. These borehole fracture data resulted in similar fracture sets (Kulatilake et al. 1998) implying that surface outcrop data represent the fracture pattern even at tunnel depth. In addition, surface outcrop data provided information on fracture trace length that was not available from borehole fracture data.

The number of fracture sets obtained for each line survey is shown in Table 1. Also the mean orientation obtained for each fracture set is given in Table 1. Note that the selected outcrops and the scanlines have different orientations. Due to the orientation sampling bias, chance of sampling fractures from different sets would be different on each of the selected scanlines. Therefore, some of the fracture sets in the studied region may not produce significant numbers on some of the chosen scanlines. Note that only line survey 22 has sampled fracture data coming from all the 4 regional fracture sets. Eight line surveys have captured only one of the 4 regional fracture sets. Six line surveys have captured 2 of the four regional fracture sets. Line survey 7 has captured data coming from 3 of the four regional fracture sets. Table 2 shows how

Regional fracture set number	Fracture set from scanline	Number of data
Fracture set I	LS 12–2	40
	LS 22–3	23
Fracture set II	LS 10–1	25
	LS 11–1	16
	LS 12–1	16
	LS 13–1	115
	LS 14–1	56
	LS 15–1	27
	LS 16–1	61
	LS 22–2	14
Fracture set III	LS 7–2	50
	LS 9–1	34

Table 2. Sorting of fracture sets appearing on the scanlines into regional fracture sets.

	LS 10–2	11
	LS 17–1	13
	LS 18–1	49
	LS 19–1	52
	LS 22–1	14
Fracture set IV	LS 7–3	19
	LS 14–2	11
	LS 17–2	18
	LS 18–2	19
	LS 19–2	14
	LS 20–1	20
	LS 21–1	25
	LS 22–4	16

the fracture sets obtained from the line surveys were sorted into the four regional fracture sets.

Goodness-of-fit of Bingham distribution (Bingham, 1964) was performed for the orientation data of each fracture set obtained from each scanline. The results show that only for two fracture sets (fracture set 2 of line survey 12 and fracture set 1 of line survey 19) the Bingham distribution is suitable to represent the statistical distribution of orientation data (see the last column of Table 1 along with the note given for *P*). For many fracture sets data were insufficient to perform the chi-square test. For the fracture sets obtained through lumped data (LS 7–22), the Bingham distribution is not suitable to represent the statistical distribution of Table 1 along with the note given for *P*).

Goodness-of-fit of hemispherical normal distribution (Kulatilake, 1985) was performed for the raw orientation data of each fracture set obtained from each line survey. The results indicate that the orientation distribution of only a few fracture sets (fracture set #3 of LS 7; fracture set #10f LS 11; fracture set #1 of LS 15; fracture set #1 of LS 17; fracture sets #1 and #2 of LS 22) can be represented by a hemispherical normal distribution (see the last column of Table 3 along with the note given for *P*). The hemispherical normal distribution for all the 4 fracture sets obtained from lumped data of LS 7–22 (see the last column of Table 3 along with the note given for *P*). The orientation variability increases with high spherical variance values and low *k* values. Accordingly, fracture set #2 of LS 7–22, fracture set #1 of LS 10 and fracture set #1 of LS 9 show the highest orientation variability.

The procedure available for correcting orientation bias (Wathugala et al. 1990, Kulatilake 1998) was applied to study the effect of orientation sampling bias on orientation distribution of fracture sets. This procedure takes into account the length of the scanline and size distribution of fractures in addition to the relative orientation between the scanline and each fracture plane. On the other hand, Terzaghi's (1965) orientation sampling bias correction only considers the relative orientation between the scanline and each fracture plane. Therefore, the procedure used to correct the orientation sampling bias is more precise than Terzaghi's (1965) orientation sampling bias correction. A difference between the raw and corrected relative frequency distributions indicates the effect of orientation sampling bias. About 5 fracture sets (LS 7–1, LS 9–1, LS 10–1 and LS 19–1) showed some significant effect of sampling bias correction (Kulatilake et al. 1998). Other fracture sets showed insignificant effect. Goodness-of-fit of hemispherical normal distribution was then performed to the corrected relative frequencies of orientation data for each fracture set (Kulatilake et al. 1990b). The results showed that for some of the fracture sets (LS 7–1, LS 9–1, LS 10–1 and LS 19–1) it is important to apply the orientation sampling bias correction in modeling joint orientation distribution (Kulatilake et al. 1998).

This study clearly shows that the available theoretical probability distributions (hemispherical normal and Bingham distributions) are insufficient for many field sites to represent the statistical distribution of orientation data. For the fracture sets that cannot be represented by a theoretical orientation probability distribution, the empirical orientation distributions obtained from the corrected relative frequency data can be used for generation of orientation values.

4 MODELING OF FRACTURE SIZE

4.1 Best probability distribution(s) to represent semi-trace length

Kolmogorov-Smirnov (K & S) goodness-of-fit test was conducted separately for semitrace length data belonging to each fracture set associated with each scanline (Table 1). For each fracture set, the semi-trace length data can be analyzed under three categories: (a) data

		Mean nor	mal vector			
Scanline number	Fracture set	Trend (°)	Plunge (°)	k	Sp. var.	Р
LS 7	1	80.34	14.68	16.75	0.0531	0.023
	2	166.75	6.62	16.49	0.0594	< 0.005
	3	256.66	64.13	11.52	0.0822	0.192
LS 8		78.54	29.09	19.60	0.0495	< 0.005
LS 9		327.28	16.97	8.98	0.1081	< 0.005
LS 10		42.40	6.04	8.45	0.1136	< 0.005
	2	337.27	28.19	10.66	0.0853	0.017

Table 3. Goodness-of-fit results of hemispherical normal distribution for orientation data.

LS11		54.31	5.05	20.37	0.0460 0.571
LS 12		55.68	14.27	14.21	0.0660 < 0.005
	2	289.44	9.51	17.29	0.0564 0.017
LS 13		196.88	10.06	10.68	0.0928 <0.005
LS 14		208.84	8.40	17.95	0.0547 0.022
	2	340.26	62.24	25.43	0.0357 0.006
LS 15	1	47.91	6.46	26.15	0.0368 0.068
LS 16	1	260.21	17.96	10.99	0.0895 <0.005
LS 17	1	146.28	28.39	30.58	0.0302 0.537
	2	329.69	70.69	66.11	0.0143 0.022
LS 18	1	157.95	20.99	20.00	0.0490 <0.005
	2	343.66	63.60	31.07	0.0305 <0.005
LS 19	1	160.49	10.31	12.32	0.0796 0.032
	2	296.12	51.49	23.78	0.0391 <0.005
LS 20	1	77.36	67.31	30.06	0.0316 0.018
LS 21	1	304.74	86.00	22.19	0.0433 < 0.005
LS 22	1	157.05	4.72	13.17	0.0709 0.322
	2	236.95	0.89	24.09	0.0396 0.155
	3	287.92	2.05	22.16	0.0432 0.019
	4	340.40	68.72	20.11	0.0466 <0.005
LS 7-22 Fol.	1	324.69	76.15	11.31	0.0878 < 0.005
LS 7-22 Jnt.	1	294.48	8.09	18.49	0.0535 <0.005
	2	224.60	4.91	5.99	0.1667 <0.005
	3	159.38	8.75	12.47	0.0798 <0.005

k=A parameter in the hemispherical normal distribution.

Sp. var.=Spherical variance.

P=Maximum significance level at which the hemispherical normal distribution is suitable to represent the statistical distribution of joint orientation data (a minimum of 0.05 is required to represent the orientation data by a hemispherical normal distribution).

Note: Upward mean normal vector of the fracture plane is considered.

above (in the case of a horizontal scanline) or to the right of scanline (in the case of a vertical scanline), (b) data below (in the case of a horizontal scanline) or to the left of scanline (in the case of a vertical scanline) and (c) data on both sides of the scanline. For the data above the scanline and both sides of the scanline, in general, the gamma distribution turned out to be the best distribution to represent semi-trace length with exponential distribution being the second best (Kulatilake et al. 1998). However, for the data below the scanline, gamma and lognormal distributions turned out to be the best and the second best, respectively (Kulatilake et al. 1998). Out of the 16 scanlines, 12 were horizontal or sub-horizontal and the rest were almost vertical. Since most of the horizontal or sub-horizontal scanlines were placed closer to the bottom than to the top of the exposure, the data above the horizontal scanlines are more reliable than the data below the scanline. Therefore, it is reasonable to conclude that gamma and exponential are the best and the second best distributions to represent semi-trace length data in general.

4.2 Corrections for trace length biases and estimation of corrected mean trace length

When the trace length on an infinite size 2-D exposure follows an exponential distribution, the semi-trace length resulting from an intersection with a scanline, placed on the same exposure follows exactly the same exponential distribution (Priest and Hudson, 1981). The relation between the actual trace length on infinite size 2-D exposure and the intersected semi-trace length incorporates the effect of the size bias associated with trace length modeling. The distribution for censored semi-trace data then follows a truncated exponential distribution (Kulatilake et al. 1993a).

Figure 4 shows a schematic of semi-trace lengths appearing on an outcrop having regular boundaries. On the outcrop, as shown in Figure 4, different censored lengths, C_i can be selected. For each C_i , the number of semi-traces less than C_i , r_i can be calculated. If the total number of semi-traces is n, then by plotting $\log_e ((n-r_i)/n)$ vs. C_i the corrected mean trace length can be estimated from the reciprocal of the slope of the plot (Kulatilake et al. 1993a). It is important to note that this procedure is valid only for the semi-traces following an exponential distribution. Once the corrected mean trace length is estimated from censored semi-trace lengths through this procedure, it is then possible to say that the trace length distribution on the infinite size 2-D exposure follows an exponential distribution with the estimated corrected mean trace length.

The reliability of the estimation produced through the above procedure depends on a number of factors. One of them is the range of values available for C_i . For an outcrop having an irregular boundary, the maximum C value available can get limited to a very small value (see Fig. 5) even though the available semi-trace length values are several times this maximum C value. To investigate this irregular boundary effect on the reliability of results, the maximum C value as well as the largest semi-trace length value were calculated for each fracture set separately for the semi-trace length data above or to the right of the scanline and data below or to the left of the scanline. The results are given in Table 4. The trace maps obtained for the scanlines were used to verify these calculated results. High largest semi-trace length and maximum C values, and high C_{max} /largest semi-trace length ratios increase the reliability of the estimated corrected mean trace length. Reliability increases with increasing correlation coefficient obtained for

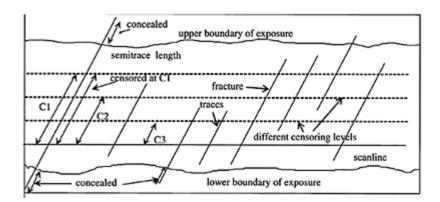


Figure 4. A sketch of semi-trace lengths censored at different censoring lengths (C_i) appearing on an outcrop having regular boundaries.

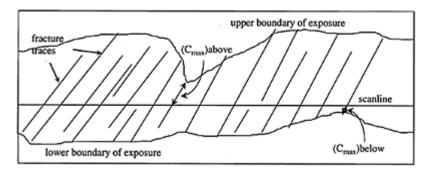


Figure 5. A sketch showing influence of irregular boundaries of an outcrop on the maximum C value.

Table 4. Summarized results for semi-trace length analysis.

ional	line fra cture	of data	imum C	be yond expo	rgest semi- T-	of lar gest	gest T- Lnth	of lar gest trace	mean T-	Corre lation. Coeffi cient	bility	rved T-	semi- T-
I, Above	12–2	40	0.73	0.45	3.96	2	4.88	2	2.51	0.9 512	L	2.10	1.28

22-3 23 0.27 0.35 4.88 2 6.10 2 3.57 0.86 L 2.05 1.05 I. eleow 12-3 40 0.43 0.73 1.68 2 4.88 2 2.25 0.8321 L 2.10 0.82 22-3 23 0.88 0.74 1.52 2 6.10 2 2.50 0.9595 L 2.05 1.00 I. how or regime 7-1 9 1.83 0.22 3.05 2 4.57 2 1.26 0.7362 L 1.16 0.46 10-1 25 0.88 0.20 3.66 2 5.18 2 0.77 0.9674 L 1.69 0.78 12-1 16 1.52 0.06 2.44 1 2.44 1 2.1 1.12 0.72 1.112 0.72 12-1 16 1.52 0.61 2 7.62 2.31 0.7687 L <t< th=""><th></th><th></th><th></th><th></th><th></th><th></th><th></th><th></th><th></th><th></th><th></th><th></th><th></th></t<>													
Below 23 23 0.88 0.74 1.52 2 6.10 2 2.50 0.955 L 2.05 1.03 The operation of the ope		22–3	23 0.27	0.35	4.88	2	6.10	2	3.57		L	2.05	1.05
Habev origin 7-1 9 1.83 0.22 3.05 2 4.57 2 1.26 0.7362 L 1.78 0.83 8 34 0.27 0.32 1.52 2 2.13 2 5.39 0.866 L 1.16 0.46 10-1 25 0.88 0.20 3.66 2 5.18 2 0.77 0.9674 L 1.69 0.78 11-1 16 - 0.94 6.40 2 7.92 2 - - L 1.52 0.51 1.21 1.51 1.21 1.22 1.22 12-1 16 1.52 0.00 2.44 1 2.44 1 0.41 1.45 0.21 1.21 1.22 14-1 5 4.24 0.23 6.10 2 7.76 2.73 0.9413 H ³ 2.80 1.61 15-1 7.7 9 0.58 0.51 2.4 6.80	,	12–2	40 0.43	0.73	1.68	2	4.88	2	2.25	0.8321	L	2.10	0.82
Above Fight Solution Soluti Solution Solution		22–3	23 0.88	0.74	1.52	2	6.10	2	2.50	0.9595	L	2.05	1.00
or No.	II,	7–1	9 1.83	0.22	3.05	2	4.57	2	1.26	0.7362	L	1.78	0.83
Indication 10-1 125 0.88 0.20 3.66 2 5.18 2 0.77 0.967 L 1.69 0.78 11-1 16 - 0.94 640 2 7.92 2 - - L 4.52 2.50 12-1 16 1.52 0.06 2.44 1 2.44 1 0.61 0.9726 L 1.22 0.72 1.21 1.22 0.72 1.21 1.22 1.22 14-1 55 0.20 0.30 0.59 2.13 2 7.62 2 2.31 0.7687 L 1.69 0.99 16-1 61 0.88 0.41 2.44 2 4.88 2 1.01 0.9678 L 1.69 0.99 16-1 61 0.88 0.49 1.52 2 4.57 2 3.28 0.8669 L 1.64 0.5 10-1 25 1.49 0.24	or												
Individual Individual One		8-1	34 0.27	0.32	1.52	2	2.13	2	5.39	0.8660	L	1.16	0.46
12-1 16 1.52 0.06 2.44 1 2.44 1 0.61 0.9726 L 1.12 0.72 13-1 115 2.41 0.21 3.96 2 5.79 1 1.55 0.9500 M 2.21 1.22 14-1 a 6 4.24 0.23 6.10 2 7.62 2 2.31 0.7687 L 1.69 0.99 15-1 27 0.30 0.59 2.13 2 3.05 2 2.31 0.7687 L 1.69 0.99 16-1 61 0.88 0.41 2.44 2 4.88 2 1.20 0.9519 L 1.71 1.06 22-2 24 0.58 0.25 4.88 2 1.96 0.9678 L 2.26 1.43 16-1 7-1 9 0.58 0.89 1.52 2 5.18 2 1.31 0.8971 L 1.60		10-1	25 0.88	0.20	3.66	2	5.18	2	0.77	0.9674	L	1.69	0.78
13-1 115 2.41 0.21 3.96 2 5.79 1 1.55 0.9500 M 2.21 1.22 14-1° 56 4.24 0.23 6.10 2 7.62 2 2.73 0.9413 H° 2.80 2.16 15-1 27 0.30 0.59 2.13 2 3.05 2 2.31 0.767 L 1.69 0.99 16-1 61 0.88 0.41 2.44 2 4.88 2 1.20 0.9519 L 1.61 0.99 22-2 24 0.58 0.25 4.88 2 6.10 2 1.96 0.9678 L 2.26 1.43 Below or fee 7-1 9 0.58 0.89 1.52 2 4.57 2 3.28 0.8660 L 1.78 0.95 Prescovert or fee 10-1 25 1.49 0.24 1.52 2 5.18 2 0.80 0.9718 L 1.60 0.82 10-1 16 0.70 0.66 </td <td></td> <td>11-1</td> <td>16 –</td> <td>0.94</td> <td>6.40</td> <td>2</td> <td>7.92</td> <td>2</td> <td>_</td> <td>_</td> <td>L</td> <td>4.52</td> <td>2.50</td>		11-1	16 –	0.94	6.40	2	7.92	2	_	_	L	4.52	2.50
14. 56 4.24 0.23 6.10 2 7.62 2 2.73 0.9413 H ^a 2.80 2.16 15-1 27 0.30 0.59 2.13 2 3.05 2 2.31 0.7687 L 1.69 0.99 16-1 61 0.88 0.41 2.44 2 4.88 2 1.20 0.9519 L 1.71 1.06 22-2 24 0.58 0.25 4.88 2 6.10 2 1.96 0.9678 L 2.26 1.43 Below 7-1 9 0.58 0.89 1.52 2 4.57 2 3.28 0.8600 L 1.16 0.70 Ideit 7-1 9 0.58 0.89 1.52 2 2.13 2 1.31 0.8971 L 1.16 0.70 Ideit 1.61 0.57 1.22 2 5.18 2 0.80 0.9718 L 1.60 0.82 12-1 16 1.07 0.66 1.07 2 <td></td> <td>12-1</td> <td>16 1.52</td> <td>0.06</td> <td>2.44</td> <td>1</td> <td>2.44</td> <td>1</td> <td>0.61</td> <td>0.9726</td> <td>L</td> <td>1.12</td> <td>0.72</td>		12-1	16 1.52	0.06	2.44	1	2.44	1	0.61	0.9726	L	1.12	0.72
1 ^a 15-1 27 0.30 0.59 2.13 2 3.05 2 2.31 0.7687 L 1.69 0.99 16-1 61 0.88 0.41 2.44 2 4.88 2 1.20 0.9519 L 1.71 1.06 22-2 24 0.58 0.25 4.88 2 6.10 2 1.96 0.9678 L 2.26 1.43 II, Below or Left 7-1 9 0.58 0.89 1.52 2 4.57 2 3.28 0.8660 L 1.78 0.95 16-1 16 0.46 0.53 1.22 2 5.18 2 0.80 0.9718 L 1.60 0.82 11-1 16 0.91 0.69 6.10 2 7.92 3.74 0.9415 L 4.52 2.03 12-1 16 1.07 0.06 1.07 2 2.44 1 0.34 0.9718 <		13–1	115 2.41	0.21	3.96	2	5.79	1	1.55	0.9500	М	2.21	1.22
16-1 61 0.88 0.41 2.44 2 4.88 2 1.20 0.9519 L 1.71 1.06 22-2 24 0.58 0.25 4.88 2 6.10 2 1.96 0.9678 L 2.26 1.43 Ingest 7-1 9 0.58 0.89 1.52 2 4.57 2 3.28 0.8660 L 1.78 0.95 Below 0.10 25 1.49 0.24 1.52 2 2.13 2 1.31 0.8971 L 1.16 0.70 10-1 25 1.49 0.24 1.52 2 5.18 2 0.80 0.9718 L 1.60 0.82 11-1 16 0.91 0.69 6.10 2 7.92 2 3.74 0.9415 L 1.60 0.82 12-1 16 1.07 0.06 1.07 2 2.44 1 0.34 0.9711 L 1.12 0.40 13-1 1.15 0.88 0.69 2.4<			56 4.24	0.23	6.10	2	7.62	2	2.73	0.9413	H ^a	2.80	2.16
22-2 24 0.58 0.25 4.88 2 6.10 2 1.96 0.9678 L 2.26 1.43 II, Below or Left 7-1 9 0.58 0.89 1.52 2 4.57 2 3.28 0.8660 L 1.78 0.95 8-1 34 0.46 0.53 1.22 2 2.13 2 1.31 0.8971 L 1.16 0.70 10-1 25 1.49 0.24 1.52 2 5.18 2 0.80 0.9718 L 1.60 0.82 11-1 16 0.91 0.69 6.10 2 7.92 2 3.74 0.9415 L 4.52 2.03 12-1 16 1.07 0.06 1.07 2 2.44 1 0.34 0.9711 L 1.12 0.40 13-1 115 0.88 0.69 2.44 2 5.79 2 2.60 0.9728 L 2.21 0.99 14-1 56 0.27 0.80 1.83		15-1	27 0.30	0.59	2.13	2	3.05	2	2.31	0.7687	L	1.69	0.99
Head bein reft 9 0.58 0.89 1.52 2 4.57 2 3.28 0.8660 L 1.78 0.95 8-1 34 0.46 0.53 1.22 2 2.13 2 1.31 0.8971 L 1.16 0.70 10-1 25 1.49 0.24 1.52 2 5.18 2 0.80 0.9718 L 1.60 0.82 11-1 16 0.91 0.69 6.10 2 7.92 2 3.74 0.9415 L 4.52 2.03 12-1 16 1.07 0.06 1.07 2 2.44 1 0.34 0.9711 L 1.12 0.40 13-1 115 0.88 0.69 2.44 2 5.79 2 2.60 0.9728 L 2.21 0.99 14-1 56 0.27 0.80 1.83 2 7.62 2 5.03 0.8660 L 2.80 0.64 15-1 27 - 0.81 1.52 2		16–1	61 0.88	0.41	2.44	2	4.88	2	1.20	0.9519	L	1.71	1.06
Below or Left 8-1 34 0.46 0.53 1.22 2 2.13 2 1.31 0.8971 L 1.16 0.70 10-1 25 1.49 0.24 1.52 2 5.18 2 0.80 0.9718 L 1.60 0.82 11-1 16 0.91 0.69 6.10 2 7.92 2 3.74 0.9415 L 4.52 2.03 12-1 16 1.07 0.06 1.07 2 2.44 1 0.34 0.9711 L 1.12 0.40 13-1 115 0.88 0.69 2.44 2 5.79 2 2.60 0.9728 L 2.21 0.99 14-1 56 0.27 0.80 1.83 2 7.62 2 5.03 0.8660 L 2.80 0.64 15-1 2.7 - 0.81 1.52 2 3.05 2 - - L		22–2	24 0.58	0.25	4.88	2	6.10	2	1.96	0.9678	L	2.26	1.43
Or Left 8-1 34 0.46 0.53 1.22 2 2.13 2 1.31 0.8971 L 1.16 0.70 10-1 25 1.49 0.24 1.52 2 5.18 2 0.80 0.9718 L 1.60 0.82 11-1 16 0.91 0.69 6.10 2 7.92 2 3.74 0.9415 L 4.52 2.03 12-1 16 1.07 0.06 1.07 2 2.44 1 0.34 0.9415 L 4.52 2.03 12-1 16 1.07 0.06 1.07 2 2.44 1 0.34 0.9415 L 4.52 2.03 13-1 115 0.88 0.69 2.44 2 5.79 2 2.60 0.9728 L 2.21 0.99 14-1 56 0.27 0.80 1.83 2 3.05 2 - - L 1.69 0.69 16-1 61 0.58 0.63 1.83 2		7–1	9 0.58	0.89	1.52	2	4.57	2	3.28	0.8660	L	1.78	0.95
10-1251.490.241.5225.1820.800.9718L1.600.8211-1160.910.696.1027.9223.740.9415L4.522.0312-1161.070.061.0722.4410.340.9711L1.120.4013-11150.880.692.4425.7922.600.9728L2.210.9914-1560.270.801.8327.6225.030.8600L2.800.6415-127-0.811.5223.052L1.690.6916-1610.580.152.4424.8820.960.9750L1.710.6522-2240.880.631.8326.1022.580.8950L2.260.83III, Above or Right7-2500.880.161.8323.3520.540.9752L1.350.60	or												
11-1 16 0.91 0.69 6.10 2 7.92 2 3.74 0.9415 L 4.52 2.03 12-1 16 1.07 0.06 1.07 2 2.44 1 0.34 0.9711 L 1.12 0.40 13-1 115 0.88 0.69 2.44 2 5.79 2 2.60 0.9728 L 2.21 0.99 14-1 56 0.27 0.80 1.83 2 7.62 2 5.03 0.8660 L 2.80 0.64 15-1 27 - 0.81 1.52 2 3.05 2 - - L 1.69 0.69 16-1 61 0.58 0.15 2.44 2 4.88 2 0.96 0.9750 L 1.71 0.65 22-2 24 0.88 0.63 1.83 2 3.35 2 0.54 0.9752 L 1.35 0.60 Above or Right 7-2 50 0.88 0.16 1.83 2		8-1	34 0.46	0.53	1.22	2	2.13	2	1.31	0.8971	L	1.16	0.70
12-1 16 1.07 0.06 1.07 2 2.44 1 0.34 0.9711 L 1.12 0.40 13-1 115 0.88 0.69 2.44 2 5.79 2 2.60 0.9728 L 2.21 0.99 14-1 56 0.27 0.80 1.83 2 7.62 2 5.03 0.860 L 2.80 0.64 15-1 27 - 0.81 1.52 2 3.05 2 - - L 1.69 0.69 16-1 61 0.58 0.15 2.44 2 4.88 2 0.96 0.9750 L 1.71 0.65 22-2 24 0.88 0.63 1.83 2 6.10 2 2.58 0.8950 L 2.26 0.83 Mabove or Right 7-2 50 0.88 0.16 1.83 2 3.35 2 0.54 0.9752 L 1.35 0.60		10-1	25 1.49	0.24	1.52	2	5.18	2	0.80	0.9718	L	1.60	0.82
13-1 115 0.88 0.69 2.44 2 5.79 2 2.60 0.9728 L 2.21 0.99 14-1 56 0.27 0.80 1.83 2 7.62 2 5.03 0.8660 L 2.80 0.64 15-1 27 - 0.81 1.52 2 3.05 2 - - L 1.69 0.69 16-1 61 0.58 0.15 2.44 2 4.88 2 0.96 0.9750 L 1.71 0.65 22-2 24 0.88 0.63 1.83 2 6.10 2 2.58 0.8950 L 2.26 0.83 III, Above or Right 7-2 50 0.88 0.16 1.83 2 3.35 2 0.54 0.9752 L 1.35 0.60		11-1	16 0.91	0.69	6.10	2	7.92	2	3.74	0.9415	L	4.52	2.03
14-1 56 0.27 0.80 1.83 2 7.62 2 5.03 0.8660 L 2.80 0.64 15-1 27 - 0.81 1.52 2 3.05 2 - - L 1.69 0.69 16-1 61 0.58 0.15 2.44 2 4.88 2 0.96 0.9750 L 1.71 0.65 22-2 24 0.88 0.63 1.83 2 6.10 2 2.58 0.8950 L 2.26 0.83 III, Above or Right 7-2 50 0.88 0.16 1.83 2 3.35 2 0.54 0.9752 L 1.35 0.60		12-1	16 1.07	0.06	1.07	2	2.44	1	0.34	0.9711	L	1.12	0.40
15-1 27 - 0.81 1.52 2 3.05 2 - - L 1.69 0.69 16-1 61 0.58 0.15 2.44 2 4.88 2 0.96 0.9750 L 1.71 0.65 22-2 24 0.88 0.63 1.83 2 6.10 2 2.58 0.8950 L 2.26 0.83 III, Above or Right 7-2 50 0.88 0.16 1.83 2 3.35 2 0.54 0.9752 L 1.35 0.60		13–1	115 0.88	0.69	2.44	2	5.79	2	2.60	0.9728	L	2.21	0.99
16-1 61 0.58 0.15 2.44 2 4.88 2 0.96 0.9750 L 1.71 0.65 22-2 24 0.88 0.63 1.83 2 6.10 2 2.58 0.8950 L 2.26 0.83 III, Above or Right 7-2 50 0.88 0.16 1.83 2 3.35 2 0.54 0.9752 L 1.35 0.60		14–1	56 0.27	0.80	1.83	2	7.62	2	5.03	0.8660	L	2.80	0.64
22–2 24 0.88 0.63 1.83 2 6.10 2 2.58 0.8950 L 2.26 0.83 III, 7–2 50 0.88 0.16 1.83 2 3.35 2 0.54 0.9752 L 1.35 0.60 Above or Right		15-1	27 –	0.81	1.52	2	3.05	2	_	-	L	1.69	0.69
III, 7–2 50 0.88 0.16 1.83 2 3.35 2 0.54 0.9752 L 1.35 0.60 Above or Right		16–1	61 0.58	0.15	2.44	2	4.88	2	0.96	0.9750	L	1.71	0.65
Above or Right		22–2	24 0.88	0.63	1.83	2	6.10	2	2.58	0.8950	L	2.26	0.83
-	Above	7–2	50 0.88	0.16	1.83	2	3.35	2	0.54	0.9752	L	1.35	0.60
9–1 ^a 34 2.74 0.38 6.10 2 7.62 2 3.84 0.9496 H ^a 3.80 2.85	Right												
		9–1 ^a	34 2.74	0.38	6.10	2	7.62	2	3.84	0.9496	H ^a	3.80	2.85

	10–2	11	1.22	0.00	1.22	1	2.13	1	0.38	0.9224	L	1.20	0.48
	17–1	13	_	0.38	4.57	1	5.18	2	_	-	L	2.97	1.17
	18–1	49	0.91	0.16	3.66	2	4.72	2	0.99	0.9239	L	1.54	0.95
	19–1	52	-	0.67	3.05	2	5.18	2	_	-	L	2.21	1.11
	22-1	14	_	0.14	1.52	1	2.44	1	_	-	L	1.11	0.45
Reg ional fra cture set	Scan line fr acture set		imum	Ratio be yond exp osure	gest semi- TLnth	Term, of la rgest semi- T	gest T-	Term, of largest trace		lation. Coef	bility	rved T-Le	
III, Below or Left	7–2	50	0.58	0.38	1.52	2	3.35	2	0.82	0.9112	L	1.35	0.76
	9–1	34	0.73	0.47	1.83	2	7.62	2	1.09	0.9379	L	3.75	0.95
	10-2	11	1.49	0.09	2.13	1	2.13	1	0.69	0.9493	L	1.20	0.72
	17–1	13	0.91	0.46	3.05	1	5.18	1	3.85	0.9126	L	2.97	1.79
	18–1	49	0.30	0.27	1.37	2	4.72	2	3.03	0.9159	L	1.54	0.59
	19–1	52	0.88	0.35	2.90	2	5.18	2	1.12	0.9230	L	2.21	1.10
	22-1	14	0.91	0.21	1.83	2	2.44	1	0.50	0.9138	L	1.11	0.66
IV, Above or Right	7–3	19	2.44	0.05	2.44	2	3.35	2	1.03	0.8959	L	0.96	0.44
	14–2 ^a	11	2.41	0.55	7.62	2	8.84	2	3.59	0.9669	\mathbf{H}^{a}	3.76	2.40
	17–2	18	-	0.78	7.62	2	9.14	2	_	_	L	4.34	3.10
	18–2	19	0.91	0.84	6.71	2	7.62	2	5.82	0.9432	L	4.64	2.98
	19–2	14	0.58	0.50	3.05	2	4.27	2	7.81	0.7071	L	2.95	1.35
	20-1	20	_	1.00	3.96	2	7.92	2	_	_	L	6.84	3.35
	21-1	25	_	0.80	6.10	2	9.14	2	_	_	L	5.43	2.66
	22–4	16	0.76	0.50	3.66	2	6.25	2	2.52	0.9134	L	3.58	2.06
IV, Below or Left	7–3	19	0.58	0.21	1.22	2	3.35	2	0.43	0.9377	L	0.96	0.52
	14–2	11	-	1.00	2.13	2	8.84	2	_	_	L	3.76	1.36
	17–2	18	-	0.56	3.05	2	9.14	2	_	_	L	4.34	1.24

18–2	19	0.58	0.58	4.11	2	7.62	2	2.36	0.9089	L	4.64	1.66
19–2	14	0.61	0.86	3.66	2	4.27	2	7.29	0.7509	L	2.95	1.60
20-1	20	_	1.00	4.11	2	7.92	2	_	-	L	6.84	3.48
21-1	25	_	0.92	4.88	1	9.14	2	_	_	L	5.43	2.76
22–4	16	1.22	0.69	3.05	2	6.25	2	2.71	0.9666	L	3.58	1.52

Note: T-Lnth denotes trace length; Semi-T-Lnth denotes semi-trace length.

Term, denotes termination: 1: Semi-trace terminates in rock; 2: Semi-trace terminates beyond of exposure. ^a The data sets which have high reliability to produce accurate estimated mean trace length.

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Regional fracture set	Scanline fracture set	Observed mean semi- trace length (m)	Standard deviation of observed semi-trace length (m)	COV of observed semi-trace length	Corrected mean trace length (m)	α	β
Ι	LS 12–2	1.28	0.91	0.72	2.51	1.95	1.29
II	LS 14–1	2.16	2.00	0.93	2.73	1.16	2.36
III	LS 9–1	2.79	2.63	0.94	3.84	1.13	3.19
IV	LS 14–2	2.40	2.07	0.87	3.59	1.34	2.88

Table 5. Summary of fracture trace length analysis results.

COV=Coefficient of variation;

 α , β =parameters of the gamma distributed trace length on infinite size 2-D exposure.

the plot between $\log_{e}\{(n-r_i)/n\}$ and C_i . A correlation coefficient value of greater than 0.9 was considered as one of the requirements to obtain high reliability. The trace map obtained for each fracture set showed what ratio of semi-traces has terminations beyond the boundary of the exposure. The higher the value for this ratio, the lower the reliability. It is considered that this ratio should be less than about 0.5 to have high reliability for the estimated corrected mean trace length. The last factor that contributes to the reliability of the estimated trace length is the number of data belonging to the fracture set. The higher the reliability of the estimated mean trace length. All the aforementioned factors were considered in assigning a rank for the reliability. The letters H, M and *L* entered in this column indicate high, medium and low reliability, respectively. It is important to note that only three data sets are categorized as high reliability data with respect to mean trace length estimation. In many cases the outcrop features (either the high irregularity or the limited extent of the outcrop) have produced low reliability data.

For the regional fracture sets II, III and IV, the probabilistic models for the trace length can be decided based on the results obtained for fracture sets LS 14–1, LS 9–1 and LS 14–2, respectively using the semi-trace length data above the respective scanlines. For regional fracture set I, all the semi-trace length data available from scanline mapping are

categorized as low reliability data. Out of the available data sets, it seems that the most reliable semi-trace length data set is the one for fracture set LS 12–2 obtained above the scanline. Therefore, the results obtained for this data set can be used to come up with a probabilistic model for fracture trace length for regional fracture set I. With that, the mean trace lengths for the regional fracture sets range between 2.50 and 3.84 m. For all four regional fracture sets, the exponential distribution can be used to represent the trace length with the estimated mean trace length values. Also, by assuming the coefficient of variation (COV) of the trace length on infinite size 2-D exposure to be the same as

Regional fracture set	Scanline fracture set	Mean diameter (m)	Variance of diameter (m ²)	α	β
Ι	LS 12–2	1.89	2.27	1.57	1.20
II	LS 14–1	0.88	1.86	0.41	2.12
III	LS 9–1	1.06	3.26	0.34	3.08
IV	LS 14–2	1.78	4.12	0.77	2.32

Table 6. Summary of 3-D fracture size (diameter) distributions for regional fracture sets.

the COV of observed semi-trace length data, the trace length in 2-D for the regional fracture sets can be represented by gamma distributions with the computed values of the parameters α and β of the distributions (Table 5), that are based on the aforementioned estimated mean trace length and the COV values.

4.3 Fracture size distribution for regional fracture sets in 3-D

For the gneissic rock mass, the fracture size in 3-D for each of the regional fracture sets can be estimated by assuming an equivalent circular disk shape for the 3-D fractures. A procedure is available in the literature to estimate fracture diameter distribution in 3-D from fracture trace length distribution on infinite 2-D exposure (Kulatilake and Wu 1986). This procedure was used to estimate the fracture size distribution for the four regional fracture sets mentioned in Section 4.2. For all the four fracture sets, the gamma distribution was found to be suitable to represent fracture diameter distributions. Table 6 provides the 3-D fracture diameter distributions obtained for the regional fracture sets of the gneissic rock mass.

5 MODELING OF DISCONTINUITY SPACING AND 1-D FREQUENCY

Goodness-of-fit tests were performed to find the suitable probability distributions as well as the best probability distribution to represent the statistical distribution of spacing for each fracture set obtained from each scanline survey. The results indicated (Kulatilake et al. 2003) that all three probability distributions lognormal, gamma and exponential are highly suitable to represent the statistical distribution of spacing for any of the fracture sets (a total of 27 fracture sets). The lognormal distribution was found to be the best distribution for 14 fracture sets and the second best distribution for 5 fracture sets. The gamma distribution turned out to be the best distribution for 7 fracture sets and the second best distribution was found to be the best distribution for 15 fracture sets. The exponential distribution was found to be the best distribution for 6 fracture sets and the second best distribution for 7 fracture sets.

The estimation of mean spacing and linear frequency (1/spacing) are based on the measurements carried out on finite length scanlines. However, unbiased estimates of these parameters should be based on infinite length scanlines. A correction was applied for this sampling bias on spacing to obtain corrected mean spacing along each scanline according to the procedure given in Kulatilake et al. (1993a). For each fracture set, the spacing distribution, including the observed mean spacing and the standard deviation of spacing is available along the scanline direction. Assuming the exponential distribution for spacing, the mean spacing corrected for spacing sampling bias for each fracture set was calculated by using the observed mean spacing and the length of the scanline (Kulatilake et al. 2003). For all the fracture sets, the length of the scanline was found to be more than 9 times of the observed mean spacing. Therefore, no difference was found between the observed and corrected mean spacing for any of the fracture sets. Now for each fracture set, the corrected mean spacing is available along the scanline direction. From the fracture set delineation analysis, the mean normal vector direction is known for each fracture set. This information was used to calculate the mean 1-D frequency along the mean normal vector direction for each fracture set according to the procedure given in Kulatilake et al. (1993a). Results are given in Table 7. For fracture set #1 of LS 13, fracture set #4 of LS 22 and fracture set #3 of LS 7, the angle between the scanline direction and the mean normal vector direction of the fracture set was found to be greater than 70 degrees (Kulatilake et al. 2003). Therefore, the reliability of the 1-D fracture frequency estimation for these three fracture sets is lower than that of the other fracture sets given in Table 7.

Table 7 shows the 1-D fracture frequency along the mean normal vector direction and the length of the scanline for each fracture set. This information was

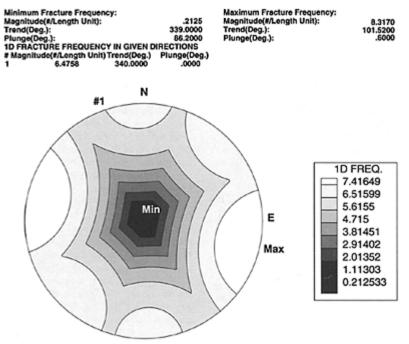
Regional fracture set	Fracture set from scanline	1-D fracture frequency along mean normal vector (# per m)	Length of scanline (m)	Weighted average 1-D fracture frequency for regional fracture set (# per m)
Joint set I	LS 12–2	8.86	9.08	6.06
	LS 22–3	2.85	7.92	
Joint set II	LS 10–1	8.23	4.66	6.30
	LS 11–1	8.50	1.83	
	LS 12–1	4.92	9.08	
	LS 14–1	6.69	9.57	

Table 7. Weighted average 1-D fracture frequencies for regional fracture sets.

	LS 15–1	9.22	2.93	
	LS 16–1	6.86	9.45	
	LS 22–2	4.69	7.92	
	LS 13–1	N/A		
Joint set III	LS 7–2	5.81	9.33	5.37
	LS 9–1	6.04	6.92	
	LS 10–2	3.05	4.66	
	LS 17–1	5.48	5.43	
	LS 18–1	5.68	9.02	
	LS 19–1	5.51	11.00	
	LS 22–1	3.67	7.92	
Foliation set	LS 14–2	2.72	9.57	3.27
IV				
	LS 17–2	3.35	5.43	
	LS 18–2	4.79	9.02	
	LS 19–2	3.35	11.00	
	LS 20–1	3.05	8.23	
	LS 21–1	2.49	10.36	
	LS 7–3	N/A		
	LS 22–4	N/A		

used to compute a weighted average 1-D fracture frequency value for each regional fracture set. In calculating the weighted average, the weight was assigned according to the length of the scanline. The obtained results are given in Table 7. Note that for this computation, the values from fracture set #1 of LS 13, fracture set #4 of LS 22 and fracture set #3 of LS 7 were not used because of low reliability.

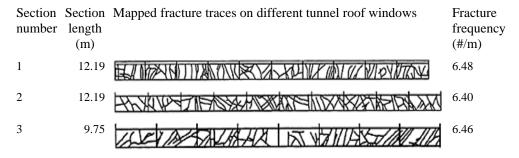
The 1-D fracture frequencies calculated along the mean normal vector directions of fracture sets for each scanline survey were then used to calculate the 1-D fracture frequency at every one degree trend and every one degree plunge in 3-D (Kulatilake, 1998). Also, the 1-D fracture frequency was estimated along the anticipated tunnel alignment (trend=340° and plunge=0°). The 1-D fracture frequencies of fracture set #1 of LS 13, fracture set #4 of LS 22 and fracture set #3 of LS 7 were not used in these calculations because of poor reliability. The calculated values are shown in Figure 6. Figure 6 provides an average



LENGTH UNIT: METER

Figure 6. 1-D fracture frequency distribution in 3-D space for the gneissic rock mass on an upper hemispherical equal angle equatorial projection.

Table 8. Fracture traces mapped on tunnel roof windows of width 0.3 m and different section lengths in gneiss and corresponding calculated 1-D fracture frequencies along the tunnel axis.



estimation of 6.5 fractures per m for 1-D fracture frequency along the tunnel axis. About one year later of this prediction, fractures traces were mapped on the tunnel roof along the alignment on segments of length 1.22 m (4 feet) and width 0.3 m (1 foot). Table 8 shows the fracture traces mapped on the tunnel roof on 4 different sections (each section having seven to ten 1.22 m length segments) in gneiss close to the field study site. These four sections have produced 1-D fracture frequencies between 6.40 and 6.48 per m (Table 8). These results show that the predicted 1-D fracture frequency along the tunnel alignment is in excellent agreement with the observed 1-D fracture frequency obtained during tunnel excavation about one year later.

6 ESTIMATION OF VOLUMETRIC FRACTURE FREQUENCY

Volumetric fracture frequency for a fracture set (number of fracture centers per unit volume), λ_{ν} , can be related to linear fracture frequency, λ_b fracture diameter *D* and fracture orientation using the following equation (Kulatilake et al. 1993a and 1996):

$$(\lambda_{\nu})_{i} = \frac{4(\lambda_{i})_{i}}{\pi E(D^{2})E(|n_{i}|)}$$
(1a)

 $E(|ni|) = E(|\mathbf{n} \cdot \mathbf{i}|)$

(1b)

where

 $(\lambda_{\nu})_i$ =volumetric fracture frequency of ith fracture set

 $(\lambda_l)_{i=linear}$ frequency of ith fracture set along the mean normal vector direction $E(D^2)$ =expected value of squared diameter

 $E(|n_i|)$ =expected value of $|n_i|$

n=unit normal vector of a fracture in the fracture set

i=unit vector along the mean normal vector (MNV) of fracture set i

All the estimated λ_{ν} values for regional fracture sets are given in Table 9.

Regional fracture set	Scanline fracture set	1-D intensity along scanline (#/m)	1-D intensity along MNV (#/m)	Mean 3D intensity (#/m ³)
Ι	LS 12–2	4.30	8.86	4.94
II	LS 14–1	6.07	6.69	4.42
III	LS 9–1	4.82	6.04	1.41
IV	LS 14–2	1.15	2.72	0.35

Table 9. Summary of 3-D fracture intensity modeling.

MNV=Mean normal vector of fracture set.

7 FRACTURE SYSTEM MODELING IN 3-D INCLUDING A VALIDATION

To describe the fracture geometry pattern in 3-D for a statistically homogeneous rock mass, it is necessary to specify the number of fracture sets, and the statistical distributions for the following fracture geometry parameters for each fracture set: (1) number of fractures per unit volume; (2) orientation; (3) diameter; and (4) location of fracture centers. Mean 3-D fracture intensity for each fracture set was estimated in section 6. Because, the fracture spacing for each fracture set follows an exponential distribution, according to the statistical theory, the Poisson distribution can be used to model the 3-D fracture intensity distribution for each fracture set with the calculated mean 3-D fracture intensity value given in Table 9. The empirical distribution obtained for the orientation was used to model the orientation distribution for each fracture set. For each fracture set, the diameter was represented by the obtained gamma distribution, according to the statistical models were used to generate the fracture system in 3-D for the gneissic rock mass.

The values used for the fracture parameters in obtaining the first generated fracture system are termed "before calibration values". The generated fracture system in 3-D was used to make predictions of fracture traces for each fracture set on a vertical 2-D window. The size of the 2-D window for each fracture set was selected in such a way to make a comparison with the fractures mapped using a scanline survey on a 2-D steep exposure that had highly irregular boundaries. For regional fracture sets I, III and IV the window length was chosen to be the same size as the length used for the scanline in the field. For regional fracture set II, the window length was chosen to be half of the scanline length used in the field. For all the four fracture sets the window height was selected to coincide with the maximum height of the exposure used in the field.

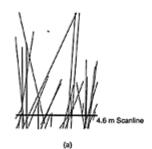
To validate the generated discontinuity system, the predicted fracture traces on selected 2-D windows from fracture sets were compared with actual field line surveys. Three criteria were used to decide whether the predicted data are in close agreement with the field data. A close agreement of the apparent orientation distribution of the chosen fracture set between the fractures mapped on the exposure in the field and the traces predicted on the selected 2-D window was considered as the first criterion. As the second criterion, the 1-D intensity value along a scanline placed around the mid level of the constructed 2-D window for the fracture set was expected to be in good agreement with the 1-D intensity value obtained along the scanline in the field survey. The third criterion used was the predicted mean trace length of the fracture set on the 2-D window should be in agreement with the range between the observed and corrected mean trace length obtained from the size analysis performed on field fracture sets the apparent orientations are in good agreement (Kulatilake et al. 1998). For some fracture sets, the diameter and the 3-D fracture intensity parameter values that were used to generate fractures had to be

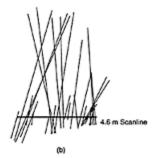
changed slightly until thegenerated fractures satisfied the aforementioned three criteria quite well. Table 10 provides modified values of the summary statistics for the diameter and 3-D intensity parameters used for the generation. These parameter values are termed the "after calibration values". Table 10 can be compared with Tables 6 and 9 to see the changes made to the fracture network model to produce generated fracture traces that compare well with the fracture traces obtained from field measurements. Figures 7a and 7c show the fractures from regional fracture sets II and IV respectively that

Regional fracture set	Scanline fracture set	Mean diameter (m)	Variance of diameter (m ²)	α	β	Mean 3-D intensity (#/m ³)
Ι	LS 12-2	2.50	2.27	2.75	0.91	4.94
II	LS 14-1	0.97	3.53	0.27	3.62	3.53
III	LS 9-1	1.06	3.26	0.34	3.08	2.83
IV	LS 14-2	1.46	4.12	0.52	2.82	0.53

Table 10. Summary statistics of regional fracture sets used for generation - after calibration.

 α , β =parameters of the gamma distributed fracture diameter.





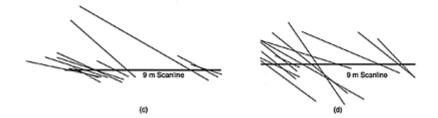


Figure 7. Comparison between predicted and field observed fracture traces on scanline LS 14: (a) predicted fracture traces of regional fracture set II on the scanline, (b) traces of regional

fracture set II obtained from field mapping on the scanline, (c) predicted fracture traces of regional fracture set IV on the scanline and (d) traces of regional fracture set IV obtained from field mapping on the scanline.

intersected a horizontal scanline that was drawn at the mid level on a vertical 2-D window that was placed in a 3-D volume used to generate fractures in 3-D. Figures 7b and 7d show the fractures from the same two sets, respectively obtained from field mapping on the same scanline. Figures 7a and 7b as well as 7c and 7d compare quite well with respect to statistical properties of fracture traces.

8 CONCLUSIONS

Four fracture sets were found to exist in the gneissic rock mass. Three of them were found to be sub-vertical joint sets. The other one is a flat lying foliation set. The mean orientation directions for these fracture sets are given in the paper. All the fracture set distributions exhibit high variability. Both the hemispherical normal and Bingham distributions were found to be unsuitable to represent the statistical distribution of orientation for most of the fracture sets. This means that if one wants to generate orientation data accurately for any of the fracture sets, then it is necessary to use the empirical distribution obtained for the orientation based on the corrected relative frequencies.

The probability distributions, gamma, exponential and lognormal were found to be highly suitable to represent the distribution of spacing of fracture sets. The estimated mean 1-D fracture frequency along the tunnel axis turned out to be about 6.5 fractures per meter. This prediction was found to be in excellent agreement with the observed 1-D fracture frequency obtained during tunnel excavation about one year later. This is one of the validations conducted for the developed fracture network.

The probability distributions, gamma, exponential and lognormal were found to be the best distributions in the given order to represent the statistical distributions of censored semi-trace length. Most of the outcrops had highly irregular boundaries. Also, the extent of some of the outcrops were not large enough to have high proportion of traces having both ends terminating in the rock or on the fractures that exist on the outcrop. These factors limited the reliability of the estimated corrected mean trace length for the fracture sets obtained through scanlines. The semi-trace length data above the scanlines for fracture sets LS 9–1, 14–1 and 14–2 were considered to produce most reliable, estimated corrected mean trace lengths. The semi-trace length data above the scanline for fracture set LS 13–1 was considered to produce estimated corrected mean trace length at a medium-level of reliability. All other semi-trace length data sets were considered to produce estimated corrected mean trace length at a low-level of reliability (see Table 4). For the regional fracture sets, the models given in Table 10 can be used to represent fracture size in 3-D and fracture intensity in 3-D.

The developed fracture network model was used to generate fractures in 3-D and then to predict fracture traces on 2-D exposures and 1-D scanlines used to map fractures. These predicted traces were used to calculate summary statistics on fracture orientation, size and intensity. These calculated values were compared with the values obtained for the same summary statistics based on the observed fracture traces appearing on 2-D and 1-D exposures that are similar to the ones used for prediction. This exercise can be considered as another validation attempt for the developed fracture network and it turned out to be successful.

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Modelling of rockbolt behavior around tunnels with emphasis on stress distribution on the rockbolt shank

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ABSTRACT: Rockbolts as a reinforcing system have significant role on the stability of tunnels. Evaluation of grouted rockbolts interaction with rock masses is of paramount interest because of their complexity. In this paper, various mathematical models of rockbolt designs are evaluated. Thereafter, performance of rockbolts as a reinforcement system is examined. In this analysis, elasto-plastic interaction of rockbolts and the rock mass, distribution of stress along mechanical and grouted rockbolts, and the effect of active and passive rockbolts on an improvement in the rock mass parametres have been modelled by FLAC software, using finite difference method. A ten-metre diametre rock tunnel was selected for analysis under various loading conditions. Rockbolts were modelled with specific diametre and spacing. The analyses were performed for one-stage and four-stage excavations and the effect of excavation method is investigated. Finally, the obtained results have been compared with a mathematical and analytical model.

1 INTRODUCTION

The main objective in design of tunnel reinforcement system is to help the rock mass to reinforce itself. One of the most common ways to reinforce the rock masses in various engineering disciplines is to use the rockbolts as a reinforcement system. This reinforcement system becomes a part of the rock mass and improves the rock mass behavioral parameters. The advantages of the rockbolts may be specified as follows:

- (a) They are applicable for any excavation shape and geometry,
- (b) They are easy to use and they can be installed fully mechanized,
- (c) They are relatively cheap,

- (d) It is possible to install them very fast and immediately after excavation,
- (e) It is possible to use them as single support or together with other support systems such as shotcrete and mesh.

These advantages result in rockbolts becoming a general reinforcement in nearly all underground excavations.

Two rockbolts groups are ungrouted mechanically and grouted rockbolts. In ungrouted rockbolts the load and deformation characteristics of the system depend upon the behavior of the anchor, washer plate and bolt head and as well as upon the behavior of the bolt shank. The rock-reinforcement interaction concepts are not difficult to use for these types of rockbolts. It is very complicated to use for grouted rockbolts and there is not a comprehensive analytical method to characterize these types of rockbolts. This is because they do not act independently of the rock mass and hence, the deformations of the rock mass and reinforcement system can not be separated. In this paper, Rockbolt behavior is modeled for both mechanical and grouted rockbolts by FLAC and the effect of rockbolt presence is investigated in different states of stress. A comparison is made between the results obtained from Convergence Control method (Indraratna & Kaiser 1991) and FLAC program (Itasca Consulting Group 1995).

2 CONVERGENCE CONTROL METHOD (CCM)

The Convergence Control method was proposed by Indraratna & Kaiser (1991) based on an elasto-plastic analysis. In this analysis, it is assumed that the rock mass is isotropic and acts elastically before yielding. Also an isotropic in-situ stress field (k_0 =1) is applied, which is more realistic for relatively deep excavations. In order to achieve plane strain condition, the amount of radial displacements is measured at a distance at least twice the tunnel radius. In this method, failure envelope is specified by Mohr-Coulomb criterion.

The strain of plastic zone is the sum of the elastic strain and plastic strain components. The elastic strain component in this zone is specified by elastic constants E and v. The plastic components may be specified by an appropriate flow rule. In this analysis the following flow rule which is pertinent to the Mohr-Coulomb criterion was used:

$$\varepsilon_r^p + \alpha \varepsilon_\theta^p = 0 \tag{1}$$

where α =the dilation factor which shows the volume change in the plastic zone. Zero volume change is presented by $\alpha=1$. $\varepsilon_r^p \varepsilon_{\theta}^p$ and are radial and tangential plastic strain respectively.

Radial strains at the tunnel boundary are the most significant quantities for determining the tunnel stability. Radial strain (ε_a^*) and convergence (u_a^*/a) at the tunnel boundary are calculated as follows after determining radius of equivalent plastic zone (R^*) (Indraratna & Kaiser 1991):

$$\varepsilon_{a}^{*} = -a(1-\nu)/2G \qquad (2)$$

$$\left\{ \sigma_{cr} M\left[\left(\frac{R^{*}}{a} \right)^{(m+a)} - 1 \right] + \sigma_{c} \left(1 - s \right) \left[\left(\frac{R^{*}}{a} \right)^{(1+a)} \right] \right\}$$

$$-\nu \sigma_{cr} / 2G \qquad u_{a}^{*} / a = (1-\nu)/2G \qquad (3)$$

$$\left(\sigma_{cr}\left\{I+M\left[\left(\frac{R^{\star}}{a}\right)^{(m+a)}-I\right]\right\}+\sigma_{c}\left(I-s\right)\left[\left(\frac{R^{\star}}{a}\right)^{(I+\alpha)}\right]\right)$$
(4)
$$M=\frac{2}{1+\alpha-\sin\phi(\alpha-1)}$$
$$m=\tan(\pi/4+\phi/2), 0< s<1$$
(5)

Figure 1 shows the variations of tunnel convergence for different patterns of rockbolts. It is observed that with increasing density of rockbolts (β) from 0.073 to 0.22, a sudden change in convergence occurs to the right side. This event occurs where the radius of equivalent plastic zone exceeds the bolt length. It occurs because the whole length of rockbolts will locate in the plastic zone.

3 NUMERICAL ANALYSIS USING FLAC PROGRAM

3.1 Theoretical aspects

Modelling by FLAC program is based on the finite difference method. In this study, the ground around the tunnel is characterized by an appropriate mesh and the corresponding force-displacement relationship is defined for it. In the constructed models, the rock mass behavior is proposed to be elasto-plastic.

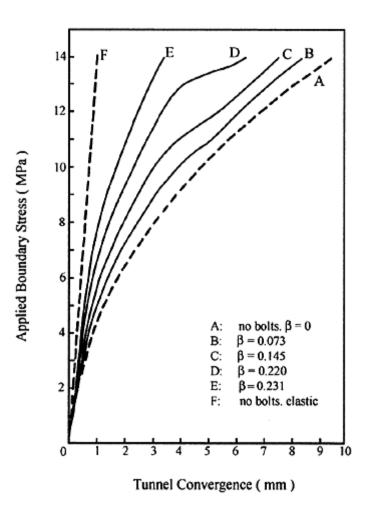


Figure 1. Effect of grouted rockbolts on convergence of tunnel.

Also the failure criterion is the linear MohrCoulomb criterion. Because the bolts are not intersected by the joints and the rockbolt is affected only by the axial force, a one dimensional model is adequate for description of rockbolt behavior.

In this model the rockbolt is divided into equal elements and the axial tension of these segments are modeled by springs having stiffness equivalent to the bolts. Interaction of rockbolts and the rock mass is modeled by series of springs and sliders. The spring stiffness is obtained from the elastic deformability of the grout and the maximum shear strength of the slider is obtained from the maximum shear strength of the grout annular in the connecting area to the rock mass. Elasto-plastic behavior of a bolt element in axial tension is illustrated in Figure 2.

The bolt stiffness (K_b) in elastic part of the graph is calculated by the following relationship:

$$K_b = E_b A / \Delta L \tag{6}$$

where E_b =modulus; A=cross section area; and ΔL =length of element. The yield stress depends

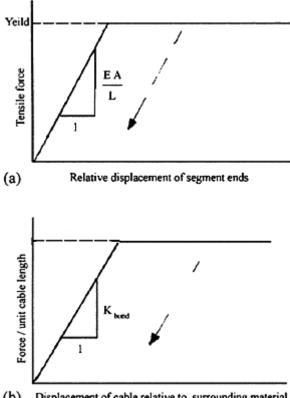




Figure 2. (a) Axial tension force-axial displacement for a steel bolt element, (b) Shear behavior of grout.

directly upon the rockbolt strength and its cross section area. After yielding, if the rockbolt is unloaded, the loading and unloading stiffnesses will be equal. Due to the relative deformation between bolt and internal area of the hole, a shear force is produced in unit length of the rockbolt. This is corresponded to the bonding stiffness (K_{bond}) which is normally obtained by pull out test. In the absence of pull out test facilities, the following relationship may be used:

$$K_{Bond} = 2\pi G_g / [\ln(1 + \frac{2t}{d_1})]$$
(7)

where *t*=grout thickness; d_1 =bolt diameter; and G_g =grout shear modulus.

The ultimate bearing capacity of the grout is obtained as:

$$P'_{ult} = \tau_{\text{peak}} \pi d_2 L \tag{8}$$

(9)

 $\tau_{\text{peak}} = \tau_1 Q_b$

where d_2 =the borehole diameter; τ_1 =nearly half of the uniaxial compressive strength of the rock or grout (the smaller one); and Q_b =a factor which depends on the quality of bonding between rock and grout. In full bonding condition, Q_b is equal to 1.

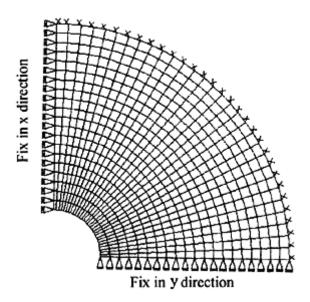


Figure 3. Computer model and boundary conditions for initial stress state.

3.2 Loading and boundary conditions

In order to determine induced stress state in the model, an analysis was carried out before excavation of tunnel when the only force on rock mass is the rock weight only. Displacements obtained in this analysis used as the initial displacements after tunnel excavation. In other words, the real displacements will be equal to the difference between displacements before and after tunnel excavation. The stresses obtained in this analysis

must also be considered as the initial stress state before tunnel excavation. For a circular tunnel and in a hydrostatic stress condition, the isotropic rock mass boundary and loading conditions is shown in Figure 3.

3.3 Discussion of results

The constructed models are in radial symmetry, as a result, only a quarter of model has been analyzed.

The rock strength parametres used in this analysis are as follows:

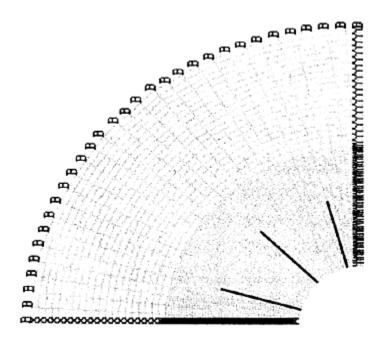
E=1.5 GPa, *v*=0.25, **Φ**=32° *c*=0.97 MPa, *T*=1.075 MPa

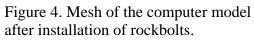
where:

E elasticity modulus *v* Poisson's ratio ϕ friction angle *c* cohesion *T* tensile strength. The analysis was per

The analysis was performed on a tunnel with 10 metre diameter and under different hydrostatic conditions. The rockbolts used have 25 mm diameter and 5.6 metre length. Numbers of 12 rockbolts are installed circumferentially with one metre spacing.

In these analyses, two types of mechanical and grouted rockbolts in two cases of tensioned and untensioned were modeled. The results obtained from the analyses, including radial and tangential stresses and the rockbolt internal tension force, are illustrated in Figures 4 to 8.





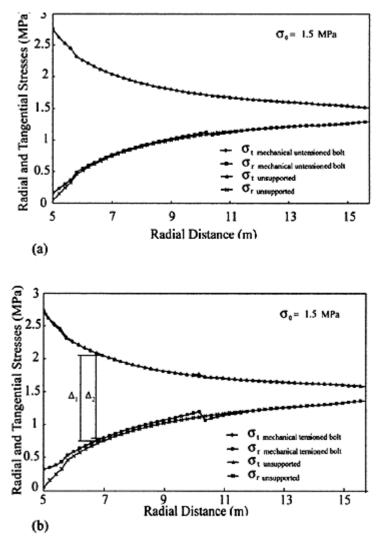


Figure 5. Stress distribution around tunnel in reinforced and non-reinforced condition, (a) Untensioned bolt, (b) Tensioned bolt.

The results obtained from the FLAC analysis show that in the case of low confining stresses, mechanical untensioned rockholts have a little influence on radial stress; however, the influence on tangential stresses is

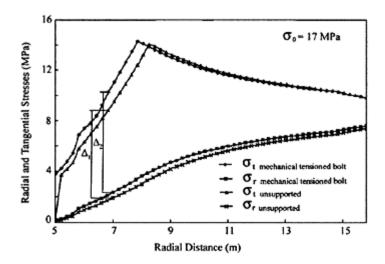


Figure 6. stress distribution around tunnel in reinforced and non-reinforced condition when $\sigma_0=17$ MPa.

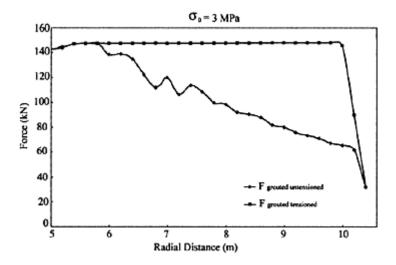


Figure 7. Variation of axial force along grouted rockbolt shank in tensioned and untensioned conditions and onestage excavation (σ_0 =3 MPa).

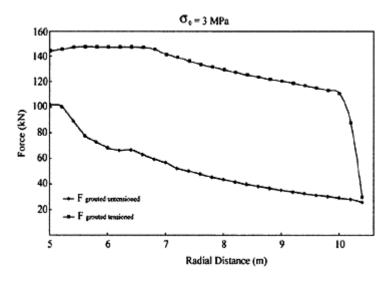


Figure 8. Variation of axial force along grouted rockbolt shank in tensioned and untensioned conditions and fourstage excavation ($\sigma_0=3$ MPa).

not significant. In this case, the deviatoric stress reduces (Fig. 5a). In Figure 5b deviatoric stresses related to conditions in which rock is reinforced by tensioned mechanical rockbolt—and non-reinforced have been shown by Δ_1 and Δ_2 respectively. Reduction of deviatoric stresses in this state is considerable. This means that the influence of tensioned rockbolts is more than untensioned ones.

In high stress levels (Fig. 6), it is observed that difference of Δ_1 and Δ_2 becomes minimum. In other words in such condition, the magnitudes of σ_r and σ_{θ} increase with together and therefore, the Mohr circles move to the right side and becoming far from the yield zone. In these stress levels, the influence of installation of rockbolts is more clear.

Comparison of the graphs corresponding to the grouted and mechanically anchored rockbolts in tensioned state show that radial stresses at the end of mechanical anchored rockbolts decrease suddenly, however, in grouted rockbolts, this stress reduction occurs gradually and smoothly. This may be included that grouted rockbolts interact more pertinent with rock masses and has a better supporting role for tunnel.

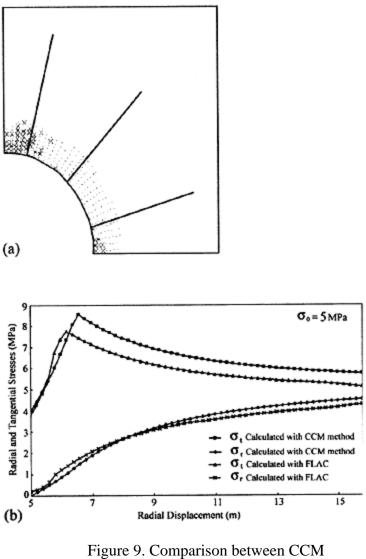


Figure 9. Comparison between CCM and FLAC models in initial stress level equal to 5 MPa, (a) Rockbolts location, (b) Stresses distribution.

In higher stress levels, the magnitudes of σ_0 increase more and radius of plastic zone decreases.

Figure 7 illustrate internal forces of the grouted rockbolt shank in 3 MPa initial stress state in two cases of tensioned and untensioned. In Figure 8, the same graph is shown in the same initial stress state when excavation has been done in four stages.

In this case, the yield length of the rockbolt is equal to 1.8 metre, whereas, in one stage excavation the yield length reaches to 4.8 metres. In such circumstances, the force in untensioned rockbolt with four-stage excavations reaches to 10 kN, whereas, in one-stage excavation, the rockbolt has yield in one metre of its length. These results show when tunnel is excavated multistage, stress distribution along the rockbolt shank will be more uniform.

4 COMPARISON OF THE TWO METHODS

In order to compare the Convergence Control method (CCM) with FLAC program, a tunnel with specified parameters was analyzed by both of methods under three initial stress conditions. The results obtained are shown in Figures 9–11.

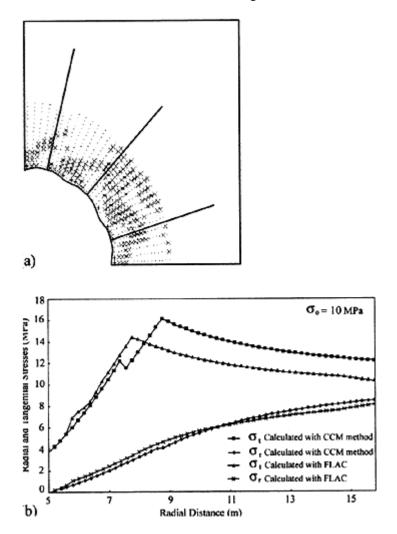


Figure 10. Comparison between CCM and FLAC models in initial stress level equal to 10 MPa, (a) Rockbolts location, (b) Stresses distribution.

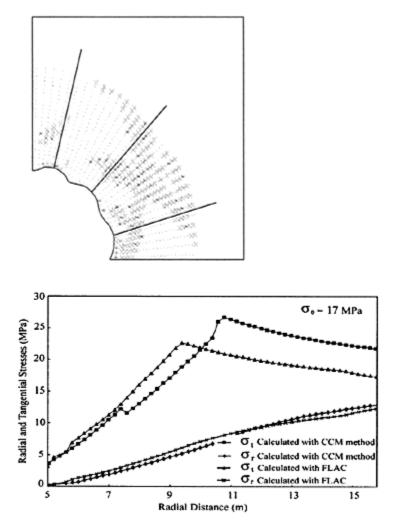


Figure 11. Comparison between CCM and FLAC models in initial stress level equal to 17 MPa, (a) Rockbolts location, (b) Stresses distribution.

Figure 9 correspond to the case in which radius of the plastic zone is smaller than the radius of neutral zone in rockbolt. In this case, the least yielding has occurred and in fact, radius of plastic zone is the free length of rockbolt.

In this case, radius of plastic zone has been obtained 7.6 metres by FLAC model and 6.544 metres by Convergence Control method. These two methods indicate a good agreement in the upward section of the σ_t plot, however, in downward section, about 0.7 MPa discrepancy is observed. Radial stresses in both methods show acceptable agreement.

Figure 10 corresponds to the case in which the radius of plastic zone is between the radius of rockbolt neutral point and the end of the rockbolt. In this condition, maximum yielding has occurred and the radius of yielding zone is equal to the total length of the anchored bolt. In this case, the radius of plastic zone obtained by Convergence Control method is equal to 8.75 metres and by FLAC method is equal to 8.2 metres. Also the radial stresses obtained by two methods have a good agreement. About tangential stresses, there is only a relatively good agreement in the upward section of the plot, while in the downward section, 1 MPa discrepancy is observed.

Figure 11 correspond to the case in which the radius of plastic zone exceeds from the reinforced section of the tunnel boundary, i.e. full yielding condition is predominant. The radius of plastic zone obtained by FLAC program in this case is equal to 9.8 metres and by Convergence Control method is equal to 10.7 metres. There is only a good agreement in the upward section of the plot corresponding to σ_r in this case.

5 SUMMARY AND CONCLUSIONS

For the purpose of analyzing grouted rockbolts behavior, a numerical method by FLAC program were used and various aspects of grouted rockbolts behavior were evaluated and compared with mechanically anchored rockbolts in both tensioned and untensioned conditions.

The results reveal the better performance of grouted rockbolts in comparison to the mechanically anchored bolts. Also it appeared that tensioned rockbolts are more effective than untensioned ones.

Comparison of the results obtained by both analytic and numeric methods shows a good agreement in determining radial stresses and also in the upward section of the tangential stress plots; however, in downward section of these plots, a discrepancy is observed and the magnitude of this discrepancy will increase by increasing of initial stress condition (e.g. depth).

It may be concluded that for better understanding and reaching a more complete solution application of two methods of analytic and numeric are necessary.

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4 In situ and laboratory testing

Changing to the Posimix4 resin bolt for Jumbo and Quick-Chem[™] at Mt Charlotte mine

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ABSTRACT: Since 1986, Mt Charlotte used the resin-anchored rebar as the main bolt type. It was installed using a purpose-built machine, and it performed very well. However in 2003, mine operational requirements led to the need for change to a jumbo-installed bolt. Assessment showed that a resin-anchored bolt would best suit the mine requirements. But as no ideal bolt was available on the market, a modification was suggested by Mt Charlotte to DSI, the manufacturer of the Posimix bolt. The new Posimix4 bolt was introduced within two months, and commissioning was successfully completed after a further four months.

The second change was to use the Jumbo to install resin cartridges in boreholes. The Quick-ChemTM resin insertion system was selected as being the most efficient and robust method available, and following minor changes it has performed beyond expectations.

Steps were taken to achieve operator acceptance of the new bolt. The major remaining acceptance issue is the remoteness of the rock bolting operator from the borehole collar, but this is being addressed. Proof-load tests confirm the new bolts perform as well as the old rebar. Overall, the bolt change process involved many issues, and was achieved in a timely and efficient manner.

1 A REVIEW OF MT CHARLOTTE BOLTING OVER THE YEARS

Mt Charlotte mine is located in a strong dolerite rock, with irregular discontinuous localscale structures forming a tightly interlocked blocky rock mass. The ground is generally good. Poor ground is associated with regional faults forming zones of more closely jointed and structured rock. The mine is fairly dry. Stress spalling and seismicity are issues, more so at depth and adjacent to large stope voids. Drive size varies from 4.5 by 4.5 m to 6 m by 5 m. In this ground the bolts retain keyblocks in place so that spans are effectively selfsupporting. Bolts are occasionally, but not frequently, required to support large rock blocks. As such, a bolt type with a reliable end anchorage is required.

Bolting varied over the 40-year history of the mine (Mikula & Lee, 2000):

- Spot bolting was applied from about 1970 in the smaller upper level drives.
- Pattern bolting, using 10 tonne mechanical shell anchor bolts and W-straps, spanned the period 1976 to 1990. However the mechanical shell bolts were subject to corrosion and became unreliable as the years passed.
- A new bolt was introduced from about 1987—a 2.4 m long, 20 tonne capacity, 20 mm diameter, resin-anchored rebar bolt (locally called the gewibar) installed in 28 mm diameter holes. Only the toe of the bolts was encapsulated.
- Variations since 1990 included cone bolts and ungrouted splitsets in seismic risk areas, and the Universal mechanical anchor bolt in the Sam Pearce Decline (linking Mt Charlotte to the Superpit) which was mined during 1997.
- From 2000, standard meshing of the backs and splitsetting of walls complied with the new Code of Practice (MOSHAB, 1999).
- Recently mine production has moved back into old areas of the mine, requiring rehabilitation of drives 20 to 30 years old. These drives have long been in stress equilibrium, so a pattern of 50% split sets and 50% rebar was adopted for rehabilitation of backs in these old drives. New development still receives 100% rebar support.
- Cable dowels (4 m and 8 m) are infrequently used, being required only where the rock mass is adversely structured and spans are large. Blasting to a stable high dome/arch shape is the preferred strategy in these difficult situations.
- W-straps are only used now at brows and pillar noses with more than one free face.
- Due to cost, shotcrete has only been sparingly used, mostly confined to key access drives that are intersected by major structures.

2 THE ORIGINAL REBAR/RESIN BOLT

To partner the introduction of the rebar bolt in 1986, a dedicated rock bolter was built inhouse by attaching a hydraulic drill boom and a HIAB basket to an old 980 loader frame (Fig. 1). All functions were controlled remotely from the basket. This allowed quality assured bolting of development (close inspection of backs, manual scaling, operator always under supported ground, fast drilling of the correct hole size, machine tightening of nuts). Underground load testing of the rebar bolt over the years showed in excess of 98% of rebar bolts achieving proof load. Mine history shows rather few instances of failed bolts. The good bolt performance is attributed to:

• Operator installing from basket close to backs—good control of bolting process and quality

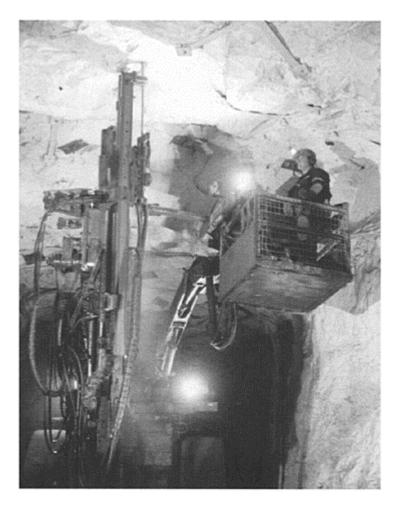


Figure 1. The 980 rock bolting machine, with drilling boom on the left controlled by the operator in the HIAB basket.

- Single pass installation with immediate 20 tonne load capacity
- Pretensioned on installation
- Manual resin insertion
- A good resin storage and transport system
- No bolt maintenance necessary
- Solid steel bolt, coarse threads—not corrosion susceptible.

3 REASON FOR THE CHANGE

The bolt type change was necessary not because of any shortfall with the rebar, but because of mine operational changes. A contract Jumbo was to be brought on site in May 2003 for a development program. This Jumbo was also to be used for rock bolting, because by this time, the mine's 980 rock bolting machine was experiencing availability problems and was to be retired after 15 years of service. However, the existing rebar was unsuitable for Jumbo installation, so a new bolt would be needed. A bolt change program was initiated covering:

- Independent review of Mt Charlotte bolting practices
- Bolt hardware duty specifications
- Assessment of new bolt system options and choice of bolt
- Underground implementation and trials
- Job Hazard Analysis for the new bolt
- Cost and operational considerations
- Performance monitoring.

4 PREFERENCE FOR A JUMBO-INSTALLED RESIN BOLT

The change to Jumbo installation was not negotiable, being an operational requirement due to production circumstances at the mine.

The rebar/resin bolting system at Mt Charlotte was proven to work well. Mt Charlotte has been successfully installing resin bolts since 1986. Common problems—resin aging, storage, handling, testing, borehole annulus to achieve proper mixing—have been properly addressed and managed, and appropriate Work Procedures are used.

The decision was made to stay with the known resin system, rather than moving into totally unknown territory with perhaps more risk of major problems and deficiencies. A small step-change meant more likely success with performance, operator acceptance, and ground control. This preference removed mechanical and cement-grouted bolts from active consideration.

Some resin-anchored Jumbo-installed bolts were available, for example the Jumbolt, Tigerbolt, Posimix, Securabolt But none were ideal for the mine conditions, experience and equipment. Reasons included variously being unable to be tensioned, having tight borehole length and diameter tolerances, and being unable to guarantee good resin mixing.

5 ASSESSMENT OF BOLT, RESIN AND JUMBO ISSUES

The particular issues at Mt Charlotte that were assessed for this project fell into three categories related to the Bolt, the Resin, and the Jumbo, as discussed below.

- Tensionable. The bolt was to be only partly encapsulated. Pretension to at least 1 tonne would be essential in the blocky rock to maintain stability and provide positive surface restraint.
- Corrosion. Must survive to the medium-term in wet conditions, and long-term in dry conditions. This would exclude bolts with fine thread components. A solid steel bolt was preferred over a hollow tube design.
- Length. A minimum 2.0 m bolt length (excluding anchorage length) for suspension and arch formation. The bolt/Jumbo configuration had to be practicable in a 4.5 m high drive, and preferably in a drive of 4.2 m. A longer version of the bolt could also be used in difficult ground.
- Load Capacity. A bolt system with a yield capacity exceeding 15 tonnes and ultimate strength about 20 tonnes or more.
- Stiffness. Preferred 5% minimum elongation at ultimate load.
- Encapsulation. Full encapsulation is not required. Debonding allows extra elongation, which suits the blocky discontinuous rockmass with shearing mainly limited to overstress (seismic) conditions.
- Energy Absorption. For the comparatively mild Charlotte seismic conditions, the bolt energy absorption should exceed 10 kJ without failure.
- A Matched System. Plates, nuts, washers must be matched to the bolt, to ensure peak performance and mobilisation of the full bolt capacity.
- Visual Indication of Load Condition. The plate/nut system must show visual signs of bolt load condition, but must not fail before the bolt fails.
- One Pass. Bolt to provide support as soon as it is installed. Excludes bolts relying on setting of cement grouts for anchorage.
- Competitive in installation speed and unit cost.

5.2 Resin issues

- Resin Annulus. Annulus thicknesses of about 3 mm are preferable. For 45 mm Jumbo Holes, the existing bolts on the market were hollow designs with about a 2 mm annulus. For these bolts the annulus tolerance is tight—too large means more resin is needed, too small means high mixing torque required. A wider annulus also has poorer rock-to-steel load transfer. In 35 mm holes, the options were the 22 mm bars with mixing aids (6.5 mm annulus if central in hole) or thicker bolts (smaller annulus, but more costly, heavier, and over-strong for the bolting pattern).
- Overdrilled Borehole Depth. Unmixed resin at the toe of the hole means that encapsulation length is less than intended, and bolt anchorage capacity could be inadequate. There was a clear preference for a 35 mm hole where the problem is less severe. Overdrill of 100 mm in a 45 mm borehole consumes 0.16 litres of resin, compared to 0.10 litres in a 35 mm borehole.
- Resin Insertion. This function needed to be carried out remotely. The conventional method of a resin cartridge mounted on the end of bolt forms a long unstable assembly—too long to install in small height headings, and suffering a high rip damage rate while trying to fit through mesh and find the borehole collar.

• Resin Mixing. Jumbo spin RPM of about 160 is low when compared to equipment used in coal mines (500 RPM). Therefore a bolt with an efficient mixing mechanism (such as the Posimix) is required, as opposed to say the Paddle Bolt.

5.3 Jumbo issues

- Jumbo Installation. The move to Jumbo installation would eliminate the intimate knowledge of installation conditions held by operators of the 980 rockbolting machine. The operator would be far from the collar, where it is hard to be sure that the process is under proper control and that bolts are properly installed.
- Large Diameter Holes. Hole diameter would increase from 28 mm to as much as 45 mm, increasing drill time and costs. While a 45 mm Jumbo hole would lead to operational simplicity (same drilling hardware to bore face and install splitsets and resin bolts), penalties included greater resin volume and tight tolerances to anchor the bolt. A 35 mm hole would require drilling hardware changes, but would be more likely to result in a better bolt performance.
- Torque. Jumbo torque is of the order of 200 Nm. This is not too much less than the 980 Rockbolting machine's 210 to 400 Nm, and better than a typical pneumatic rattle gun at 100–200 Nm, or an airleg at less than 150 Nm. As the Jumbo could potentially overmix the resin, Procedural controls would be needed. Jumbo torque was expected to achieve about 2 to 3 tonnes pretension on a rebar with
- coarse thread. Split-Feed Booms. Split-feed booms allow installation of these bolts in the typical 4.5 m high headings. A fixed-boom Jumbo would be unable to install these bolts perpendicular to the backs.

6 POSIMIX4—THE SELECTED BOLT

No available bolt was considered to meet the requirements. However, Garford Pty Ltd had just developed the "Dynamic" bolt in Western Australia in association with Dywidag-Systems International (DSI—Davies, pers. comm.). That bolt was similar to the "Posimix" bolt, but with a 6 mm thick 38 mm pitch spiral spring in place of the original thin Posimix wire, for resin installation into 43 mm holes. A laboratory mixing trial of the Dynamic bolt had shown that the mixing of the resin in the annulus was excellent. The anticlockwise rotation of the Right Hand lay spiral spring forced the plastic resin cartridge packing to the back end of the borehole.

The option of adapting the old Posimix bolt (developed for 32–33 mm holes) with a similar thicker spring, to suit installation in 35 mm diameter holes, was conceived. This occurred during a site review of bolting practices (Thompson, 2003) combined with a trial of the original Posimix bolt, carried out on the 1270 ramp at Mt Charlotte on 11th April 2003. It was apparent that the bigger spring would be a robust engineering solution to deal effectively with the large annulus, improving resin mixing and load transfer properties.

On this basis, Mt Charlotte approached DSI with the request to manufacture the new bolt design for an immediate trial. The Posimix4 was duly fabricated and introduced on the 1508 Level in the mine on 23rd May 2003.

6.1 Description of Posimix4 bolt system

The Posimix4 system at Mt Charlotte comprised:

- Drilling system—35.2 mm bit, 3.1 m R32/R25 rod on R38/R32 coupling
- 20 mm rebar 2.4 m long
- Spiral spring, RH lay, 38 mm pitch, 4.5 mm wire, around top 500 mm of bar (Fig. 2)
- Minova Toospeedie resin cartridge, 1.2 m long, 26 mm dia, 50/50 Xfast/slow set, with QuickChemTM cap (Fig. 2)
- Resin annulus variable up to 6.5 mm
- Domed nut with plastic torque insert in nut, 95 ft-lb minimum breakout
- Bearing plate 150 mm square by 6 mm, 30 mm hole.

6.2 Posimix4 compared to the original Posimix bolt

The Posimix4 is improved over the original Posimix for several reasons:

- Shorter thicker spring (new pitch 38 mm compared to the old 55 mm) gives operators more control, more robust to insert through mesh, smaller annulus allows use of a shorter length spring
- Aggressive spring design means resin is always mixed well even with short mixing times, except perhaps at the very toe of hole
- Bolt is always central in the hole—constant annulus improves load transfer capability
- Spin torque while mixing resin is increased due to the large spring, so slow set resin is used other than at the toe of the hole
- Borehole wall and bolt surface areas, and hence resin bonding areas, are the same or increased.

Some data for the new Posimix4 bolt compared with the old rebar bolt are shown in Table 1.

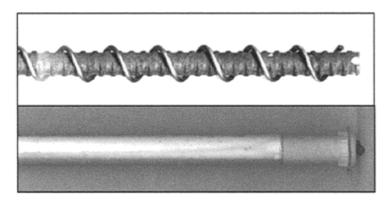


Figure 2. Upper—The end of the Posimix4 bar with the RH lay spring. The spring wraps around only the top

500 mm of the 2.4 m long bar. Lower—The end of the resin cartridge showing the Quick-ChemTM cap.

Table 1. Data for the new Posimix4 bolt and the old rebar bolt previously used at Mt Charlotte.

Item	New posimix4	Old rebar
Bolt description	20 mm rebar with spring welded to top 500 mm	20 mm rebar
Borehole diameter	35 mm	28 mm
Resin volume per installation	0.59 litres	0.23 litres
Unmixed resin in a hole overdrilled by 100 mm	0.10 litres	0.06 litres
Hardware cost of one bolt/nut/plate/resin	\$18	\$14

7 QUICK-CHEMTM___THE SELECTED RESIN INSERTION SYSTEM

A reliable cartridge insertion system was required. A site trial of the Quick-ChemTM Resin Insertion System (Mclaren, 2003) was done (at the 1270 ramp), as it was thought to offer the best solution for resin insertion in 35 mm diameter boreholes (Fig. 3). The system comprises:

- 2.5 m long polycarbonate tube (26 mm ID, 32 mm OD) with standard drill rod fitting to push resin cartridges to end of hole.
- 26 mm ID plastic cap, 38 mm outer edge, to retain resin inserted in a 35 mm hole (Fig. 4).

Quick-ChemTM was observed to be a good way to achieve the following:

• No ripped or burst resin in the hole.

• Resins pulled up to the toe of the hole inside the plastic insertion tube. This was robust and reliable, and eliminated problems with other methods which push resin up holes, causing damage in attempting to locate the borehole collar, push through mesh, or push past broken ground.

- Able to insert resin intact through broken ground-copes well with non-perfect ground.
- Resin positively placed to the end of the hole, not part way in.
- No person working under unbolted ground.

Observations also led to several cautions with the Quick-ChemTM system:

• It can be very hard to align the resin inserter tube with a hole that is hard to see (due to uneven ground).

Procedures (paint hole collars, align booms) may help if a spotter is not available.

- Caps to be on the correct end of dual fast/slow resins.
- The insertion tube must be run to toe of hole.
- Insertion tubes can be damaged—1 tube per 100 bolts initial wastage rate.

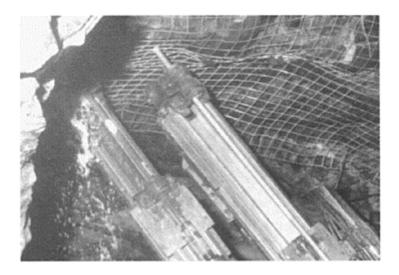


Figure 3. The Quick-Chem[™] inserter tube is mounted on the right-hand Jumbo boom, ready for insertion of resin as soon as the left-hand boom has completed drilling the hole.

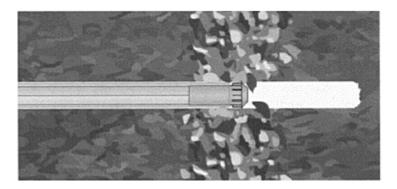


Figure 4. Schematic of the Quick-ChemTM system. The resin cartridge within the plastic inserter tube is being carried through a zone of broken ground to be positively located at the end of the borehole.

8 TRIAL OF THE NEW POSIMIX4 BOLT

There was a certain element of bravado in committing to a new bolt before it had even been manufactured or tested. However, a commitment to the bolt was considered to be correct for these reasons:

- It suited a 35 mm hole
- Resin was trusted, and the resin insertion system seen to work for 35 mm holes
- It was based on the 22 mm rebar which was also known and trusted
- The spring concept was known to work for the 45 mm Dynamic bolt
- This bolt would be a step-change from the existing method, not a completely new system.

The contractor Jumbo, however, was on site and ready to work before the new bolts were fabricated. Thus for two weeks the original design Posimix bolt was used until the new bolt was available.

The owner of the Quick-ChemTM technology was contracted on site and demonstrated the resin insertion method to the operators. The contract Jumbo operators were skilled at rock bolting using their split-feed Jumbo, and trained the local mine operators. For the two-boom split-feed Jumbo with off-sider/spotter, the bolting and meshing procedure was soon settled as follows:

- Drill one splitset hole, and pin the sheet of mesh to the backs with the splitset.
- Set up one boom to drill Posimix4 holes with a 35 mm bit.
- Set up second boom with Quick-ChemTM dolly and tube.
- Drill first hole for bolt. While this is happening, prepare and position Quick-Chem[™] boom close and parallel to drilling boom (refer Fig. 3).
- Move the drilling boom over to start drilling the second rebar hole.
- While boring, index Quick-Chem[™] boom over to the previous hole collar and install the resin.
- Repeat drilling holes, and placing resin in each hole as it is completed.
- When the row of holes is prepared, remove QuickChem[™] hardware and replace with the Posimix4 bolt installation spinner.
- Position a Posimix4 bolt in the drifter.
- Align the bolt end with the borehole (offsider assisting) and install bolt.
- Ensure work is well organised so that swaps between R38 and R32 hardware are minimised.

Additional steps are required for operating without an offsider and/or with only one split-feed boom. In low height drives, say below 5 m, a suitable Jumbo is required, with either split-feed booms and/or short feed rail lengths.

9 FIELD AND LABORATORY PERFORMANCE AND TESTING

Field testing has shown that bolt anchorage is adequate. Bolt testing to a 12.5 tonne proof load was achieved in 100% of the original Posimix bolts tested, and in 96% of the Posimix4 bolts (the one inadequate bolt of the latter type was installed in poor ground in the 1653 area, and slipped at 9 tonnes).

Serendipitously, a Posimix4 bolt (Fig. 5) was recovered two days after installation, when for other reasons it was decided to blast down an area of backs that had just been bolted. The bolt was central in the hole, the resin was completely mixed, solid, and without odour or voids. The plastic sleeve did not glove the bolt, rather it was wrapped around the toe 110 mm of the bolt. The rough resin/rock interface was reflected in the surface roughness of the hardened resin.

The bolt manufacturer (DSI) has provided three Laboratory test sequences:

- Resin Mixing Test (Rataj, 2003a): The Posimix4 bolt was set up in a lathe and spun into a 44.5 mm ID plastic tube (a 35 mm tube was not available). The test video shows the plastic pushed up the tube and the excellent mixing achieved. The completed sample shows uniform colour achieved even in the wide tube.
- Resin Mixing Test Report—Thread Bar and Old Posimix Bar (Rataj, 2003b): A rebar and the original Posimix bar were spun into 35 mm diameter steel tubes. The Posimix achieved good resin mixing. However the plain threadbar did not, leaving some unmixed (soft) resin still partly in the plastic cartridge.

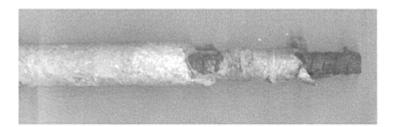


Figure 5. Toe 200 mm of a Posimix4 bolt recovered two days after installation. The resin cast was well mixed and shaped to the borehole. The plastic sleeve is wrapped around the right-hand end (toe end) of the bolt.

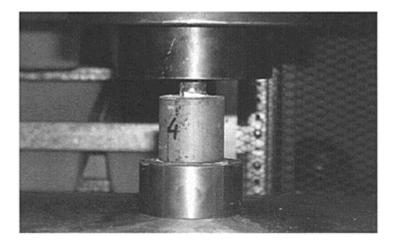


Figure 6. Test cylinder 50 mm long containing a bar sample in resin, mounted for push testing.

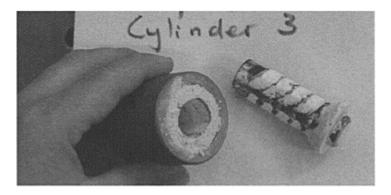


Figure 7. Push test result showing separation at the barresin interface.

Table 2. Comparison of push test peak loads for several bolt types.

Bolt system	Average peak load (kN)
Posimix bar in 35 mm steel tube	49
Tiger bolt in 45 mm steel tube	40
AX bar in 27 mm steel tube	70

Push Tests—Rebar (Rataj, 2004): Bar samples were resin-grouted into 50 mm long cylinders. The cylinders were mounted for push testing (Fig. 6). Separation at the barresin interface occurred at an average peak load of 49 kN (Fig. 7). Comparison with other bolts is shown in Table 2. Extrapolating, the bolt ultimate tensile strength of 200 kN would be expected for about a 200 mm long encapsulation length. The function of the thick Posimix4 spring appears to be dominantly mixing, bolt centralising, and pushing the plastic sleeve to the toe of the hole.

10 FIELD COMMISSIONING OBSERVATIONS

There were commissioning issues with the bolt, but all of these were satisfactorily resolved in the first eight weeks of the new bolt. The decision was since made to convert one boom of a second Jumbo to split-feed, to enable Posimix4 bolting with that machine as well. Commissioning observations and issues were as follows.

10.1 Bolt installation issues

- Nuts rattling from ends of bolts in the drifter. A nut must be positioned on the Posimix4 bolt so that it can be assembled in the drifter for installation. The coarse-thread nuts were rattling off the bar while bolts were being inserted. This was solved by siliconing the nuts on. Bolts now come with nuts pre-attached.
- Bolts falling from drifter during installation. The short spring typically passed easily through two layers of mesh, and was sufficiently robust to not be damaged on the mesh or the borehole collar. However, a common issue is that bolt assemblies fall from drifters if the bolts get badly caught in mesh. This is more likely if the operator cannot see the hole collar from the cab due to the shape of the backs, if there is no offsider to assist spotting into the hole, and if the boom and the borehole axis are misaligned. The bolt has to be reloaded into the drifter to try again.
- Damage to bolts during insertion. Damage is rare. Occasionally a damaged spring is seen. The heavy gauge wire is not springy, and welds do not detach from the bar.

10.2 Resin insertion issues

- Quick-ChemTM caps slipping off the resins. The caps are designed for a 26 mm resin cartridge. However the initial TooSpeedie resin cartridge came in a 25 mm package, and it was found that the caps tended to slide off the resins. This was solved by using longer Quick-ChemTM caps, and increasing the resin diameter to 26 mm.
- Resin insertion and quality control. The Quick-Chem[™] resin insertion system
 performed very well, inserting the resin to the back of the hole every time. One
 procedural issue was to ensure the resin cartridge entered the hole with the fast-set end
 leading. No damaged resin cartridges have been observed. The resin inserting tube
 suffers wear and tear from impact with the rock, and has a life of about 100 insertions.
- Aligning Quick-ChemTM resin inserter tube and drifter carrying bolt with the borehole. Sometimes it is difficult to align the resin inserter tube and later the bolt with the borehole collar, although adhering to a procedure can assist. This issue needs a better

solution, probably to be found in modern smart technology. A good solution could be defined as one that negates the need for an offsider even if the borehole collar is not visible to the operator.

10.3 Resin mixing and setting issues

- Jumbo spin motor anti-jam function. Initially a medium set resin was used. However because of the aggressive spring design on the bolt, this resin was too viscous during mixing and the Jumbo spin motors were backing off at high torque as the anti-jam function came into play. This function had to be disabled.
- Drive nut torque rating exceeded. With the Jumbo spin motors now acting to capacity, the mixing torque required sometimes exceeded the breakout force on the nut. Unseen by the operator, the nut would run up the bar leaving long tails, and incompletely mixed resin would have poor strength. This was solved by moving to a less viscous (slower set) resin formulation. This shows that the Jumbo spinner motors are sufficient for the spinning stage, and the spring does not cause fouling inside the borehole.
- Resin spin and hold times. To address the drive nut torque issue, the resin was changed to a TooSpeedie 40/60 fast/slow set. However this required a long 'hold' time (exceeding 30 seconds) before the bolt could be tensioned. A final change to 50/50 extra-fast/slow set resin was made, and has given good performance since. It appears that in the 40/60 formulation some of the slow resin was mixing with the fast set, increasing the set time. The resin manufacturer advice is that the Approximate Spin and Minimum Hold times are affected by temperature, mining conditions, equipment, annulus, resin age, and resin storage conditions (Minova Australia Pty Ltd, 2003). It is necessary to evaluate optimum resin spin/hold times for each site.
- Resin anchorage insufficient. Occasionally, a bolt would not take load. The causes were found to be either:
 - poor ground, with resin leaking into fractures and voids around the borehole, or
 - not enough encapsulation. This occurs if the hole is overdrilled, or the backs are rough, or the mesh is not tight to the backs, so that the bolt cannot be installed all the way to the back of the hole. Operator care was needed to drill holes no longer than 2.3 m, to avoid dead resin at the end of the hole. The 3.1 m long Jumbo steel used can drill to 2.6 m if not terminated short. The next shorter Jumbo steel available is 2.7 m long, and can only drill 2.2 m long holes.

In both cases the end result is that encapsulation is insufficient, and/or is only in the slow set resin. The solution is to reinstall with another resin cartridge. The increase to the 26 mm diameter resin cartridge helped to minimise the likelihood of this. For the same reason the 1.2 m cartridge length has been retained.

10.4 Installation rate

• Installation rate was seen to be sensitive to operator skill, and increased with time and experience with the system. The rate also increased if an offsider was available to alternate the hardware (drilling, splitsets, Posimix4, resin), assemble bolt hardware in

the drifter, retrieve and reset bolts dislodged or fallen during the installation process, and spot hardware into boreholes.

• The Posimix4 installation rates achieved by an experienced operator ranged from about 10 per hour consistently, to 15 per hour when everything goes well.

11 FURTHER DEVELOPMENTS

The major remaining acceptance issue is the remoteness of the rock bolting operator from the borehole collar. The difference between the old 980 Rock Bolting machine, where the operator was a metre away and in close control of events, and the Jumbo, where the operator is sometimes unable to even see the borehole collar, is evident. A technical solution (a video camera on the Jumbo boom) is being trialled.

A 3.4 m long Posimix4 bolt is also used in the mine in difficult ground, such as in rehabilitation after falls. The 3.4 m capability gives the mine rapid and easy response, with immediate support on a hole-by-hole basis. This contrasts with cable bolting, and also avoids delays while waiting for grout to cure.

Differences for the 3.4 m bolt compared to the 2.4 m installations are:

- a 3.7 m long R32/R25 drilling steel is used,
- a 2.4 m long 26 mm diameter Medium/Slow set resin cartridge is used for added encapsulation.

The same 3 m long resin installation tube is used, with a little Jumbo water pressure added to push the resin the final short distance to the toe of the longer borehole.

12 CONCLUSIONS

The need for change from one rock bolt to another at Mt Charlotte was imposed by mine operational requirements, rather than by any shortfall in the existing bolting system. Despite the plethora of bolts on the market, no bolt was ideal for Mt Charlotte circumstances.

Therefore, to achieve the objectives a technical development was suggested by Mt Charlotte to the Manufacturer, DSI, modifying the existing Posimix bolt to the Posimix4. Working with DSI, the new bolt was introduced within two months, and commissioning was largely complete after a further two months.

The Quick-ChemTM resin insertion system was selected as being the most efficient and robust method of installing resin cartridges in the boreholes, and following minor changes it has performed beyond expectations.

Operator acceptance of the new bolt was largely achieved because of the following:

- The change to the new Posimix4 was only a step-change in bolt model rather than a foreign bolt.
- The owner of the Quick-ChemTM technology was available to train the operators.
- The contract Jumbo operators provided training in installation using their split-feed Jumbo.

- The technical issues with resin cartridges were solved within a few weeks.
- The bolts, when installed, performed as well as the old rebar, as confirmed by proofload tests.

Overall, the process of change of bolt involved a wide range of issues. Through the cooperation of the parties involved, it was achieved in a timely and efficient manner within the operational requirements of the mine. To sum it up: "Change happens—after all, mining is a dynamic industry, isn't it?"

ACKNOWLEDGEMENTS

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Bolt surface profiles—an important parameter in load transfer capacity appraisal

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ABSTRACT: The role of bolt surface profiles on load transfer mechanism of bolt/resin/rock is examined in the laboratory experiments under both constant normal stiffness (CNS) and constant normal load (CNL) conditions. The CNS tests were carried out in a specially constructed shearing apparatus, while the CNL tests were carried out using the short encapsulation push and pull tests in steel tubes. The CNL tests included tests on smooth walled and rough surfaced bolts. Further studies undertaken included a series of both underground and laboratory conventional pull tests on short length resin anchors (up to 300 mm long). The study have demonstrated that; load transfer capacity of bolt is influenced by the method of testing; the bolt profile configuration played a dominant role influencing the load transfer capacity of the bolt resin rock interaction; and increased thickness of resin encapsulation had a negative influence on bolt anchorage performance.

1 INTRODUCTION

In the market today there is a variety of different rock bolt designs deployed for strata reinforcement. These rock bolts vary in appearance, based on the way they are manufactured, and how they are to operate in strata support applications. The basic resin or grout anchored rock bolt consists of a solid steel bar with some form of rib or thread profiles hot rolled onto the outside of the bar, as well as a nut and a thread at one end of the bar to enable the nut to be tightened up against the bearing plate and rock face. Irrespective of the bolt type, it is this surface profile that plays a major influence on the effective functioning of the bolt as it influences the load transfer mechanisms between rock, resin and rock bolt.

Currently there are two common methods of assessing the load transfer capability of bolt, the well-known conventional short encapsulation pull out test, and short length push test (Fabjanczyk and Tarrant, 1991). Both tests are conducted under constant normal load

condition, which is applicable to shearing across planar and regular surfaces whereby the process of shearing does not produce any noticeable vertical displacement across the shearing surfaces. Thus, both systems of testing ignore the additional forces generated due to vertical displacement of the resin during the shearing process due to bolt ribs. The results of these tests can be influenced by such factors as the annulus thickness of the resin encapsulation and improper mixing of the resin in the hole, commonly known as gloving.

Two methods of assessing the load transfer capacity of bolts under two different environments are discussed. The first method describes the CNS method of evaluating the load transfer mechanism of bolt and the second method deals with the short encapsulation test under both the pull and push testing of bolts in steel sleeve. A comparison of the validity of each test method is discussed in light of the bolt surface roughness conditions. Table 1 lists the detail of various bolts used in the study. Because of the nature of the technique, only two bolts were tested under CNS conditions. All the tests were conducted with resin anchored rock bolts and is not concerned with point anchored or friction anchored rock bolt system.

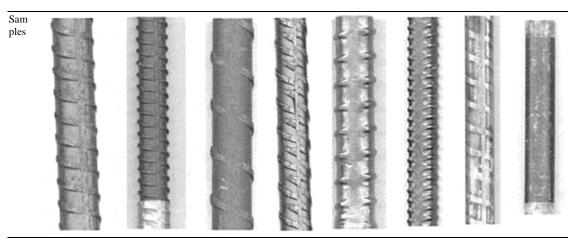
2 THE CNS TECHNIQUE

The CNS technique consists of shearing a flattened bolt surface against a resin printed image of the bolt surface profiles in a specially constructed CNS testing machine.

Figure 1 is a general view of the CNS testing apparatus used for the study Details of the equipment setup is described elsewhere (Aziz, Dey and Indrartna, 1999) and (Aziz, 2002). Briefly, the equipment

T1	T2	T3	T4	T5	T6	S 1	S 2
						(Ro	(Smo
						ugh)	oth)
Profile	12.00	12.00	25.00	12.0	12.0	8.0 -	_
centres	mm	mm	mm	0	mm	mm	
				mm			
Profile	1.00	1.60	0.80	1.50	1.24	1.5	1.6
height	mm	mm	mm	mm	mm	mm <0.	mm
						1mm	
Profile angle	22.5°	22.5°	22.5°	19°	4.8°	8.8°-	_
Profile	1.50	2.00	2.50	1.80	1.6	2.0-	_
top width	mm	mm	mm	mm	mm	mm	
Profile	3.00	3.50	5.00	3.70	3.8	3.5 -	-
base width	mm	mm	mm	mm	mm	mm	

Table 1. Rock bolt specification.



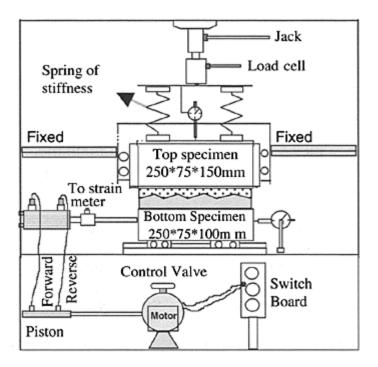


Figure 1. Schematic diagram of CNS apparatus.

consisted of a set of two large shear boxes to hold the samples in position during testing. The size of the bottom shear box was $250 \times 75 \times 100$ mm while the top shear box is

 $250 \times 75 \times 150$ mm. A set of four springs were used to simulate the normal stiffness of the surrounding rock mass.

The preparation of the shearing components was as follows. A 100 mm length of a bolt was cut and then drilled through. The hollow bolt segment was then cut along the bolt axis from one side and preheated to open up into a flat surface as shown in Figure 2. Theflattened surface of the bolt was then welded on the

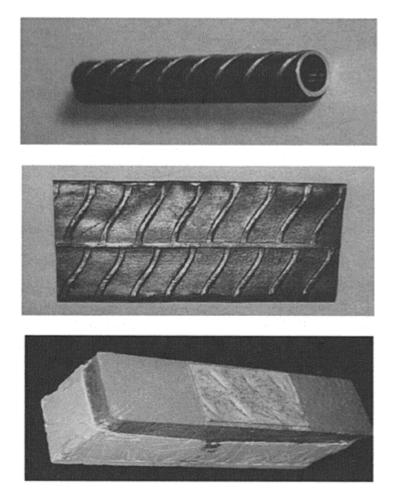


Figure 2. (a) A hollow bolt section then (b) flattened surface and (c) the image cast of the bolt surface.

bottom plate of the top shear box of the CNS testing machine.

Only two bolt Types were subjected to CNS testing. They were Type I and Type III bolts respectively. The welded bolt surface on the bottom plate of the top shear box was

used to print the image of bolt surface on cast resin samples. For obvious economic reasons, the samples were cast in two parts. Nearly three-fourth of the mould was cast with high strength casting plaster and the remaining one-fourth was topped up with a chemical resin commonly used for bolt installation in underground coal mines. The properties of the hardened resin after two weeks were, uniaxial compressive strength (σ_c)=76.5 MPa, tensile strength (σ_t)=13.5 MPa, and Young's modulus (E)=11.7 GPa. The cured plaster showed a consistent σ_c of about 20 MPa, σ_t) of about 6 MPa, and E of 7.3 GPa. The resin sample prepared in this way matched exactly with the bolt surface, allowing a close representation of the bolt/resin interface in practice.

3 SHEAR BEHAVIOUR OF BOLT/RESIN INTERFACE

3.1 Effect of normal stress on stress paths

Figure 3 shows the shear stress profiles of the bolt/resin interface for selected normal stress conditions for Bolt Type T1. The difference between stress profiles for various loading cycles was negligible at low values of σ_{no} (Figure 3a). This was gradually increased with increasing value of σ_{no} reaching a maximum between 3 and 4.5 MPa (Figure 3b). Beyond a 4.5 MPa confining pressure, the difference between stress profiles for the loading cycles 1 and 2 decreased again (Figure 3c). A similar trend was also observed for the Bolt Type T2 surface (not shown in the figure).

3.2 Dilation behaviour

For the first cycle of loading, Figures 4a and 4b show the variation of dilation with shear displacement at various normal stresses for Bolts Type T1 and T3 respectively. For various values of σ_{no} , the maximum dilation for both Bolt Types T1 and T3 occurred at a shear displacement of 17–18 mm and 7–8 mm respectively. Clearly, when compared with the rib spacings on each bolt type, it is reasonable to conclude that the peak dilation occurred at a shear displacement of about 60% of the bolt rib spacing.

3.3 Effect of normal stress on peak shear

Figures 4c and 4d show the variation of shear stress with shear displacement for the first cycle of loading at various normal stresses, for Bolts Type T1 and T3 respectively. The shear displacement for peak shear stresses, for both bolt types, increased with increasing value of σ_{no} . However, there was a gradual reduction

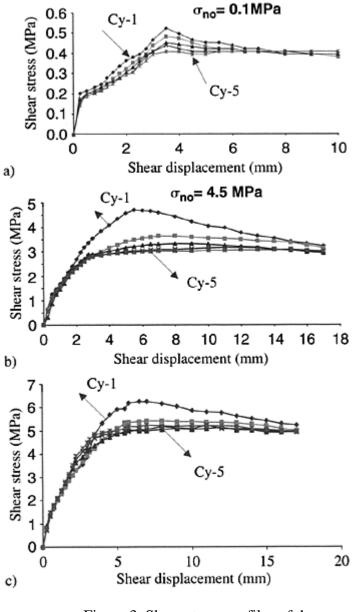


Figure 3. Shear stress profiles of the Type T1 bolt from selected tests.

in the difference between the peak shear stress profiles with increasing value of σ_{no} . The shear displacement required to reach the peak shear strength was considered to be a function of the applied normal stress and the surface properties of the resin, assuming that the geometry of the bolt surface remained constant for a particular type of bolt.

3.4 Overall shear behaviour of Type I and Type II bolts

Figure 5 shows the shear stress profiles of both Bolt Types T1 and T3 respectively for the first cycle of loading. The following observations are noted:

- The shear stress profiles around peak values are similar for both bolt types. However, slightly higher stress values were recorded for the Bolt Type T1 at low normal stress levels, whereas slightly higher stress values are observed for the Bolt Type T3 at high normal stress levels.
- Post peak shear stress values are higher for the Bolt Type T3 indicating better performance in the post
- peak region. Shear displacements at peak shear are higher for Bolt Type T3 indicating the ability of the bolt to

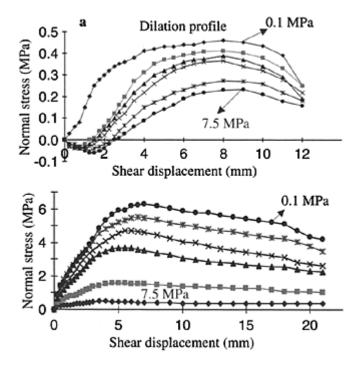


Figure 4. First loading cycle dilation and shear stress profiles for both Bolt Types T1 and T3.

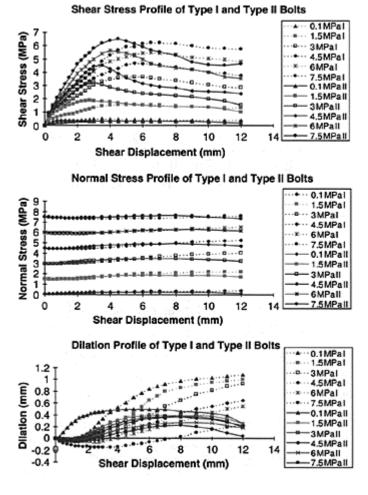


Figure 5. Comparison of stress profile and dilation of Bolt Types T1 and T3 for first cycle of loading.

accommodate a relatively greater rock displacement before instability stage is reached.

• Dilation is greater in the case of Bolt Type I.

4 CNL—SHORT ENCAPSULATION TESTING

The short encapsulation push test is considered as a quick method of assessing the load transfer capacity of bolt resin interface. Tested in a short steel tube the method has, for many years, been considered less than satisfactory as concerns have been expressed about

the methodology of testing, in which the bolt is pushed out of the steel tube rather than being pulled out. In reality the installation and subsequent performance of bolts in-situ results in the bolt being in tension and sometimes in tension and shear. There will be a general reduction in bolt cross section as a result of bolt tensioning, causing premature bolt resin surface contact failure and loss of grip. Accordingly, two methods of shearing of bolt by both pull and push test are examined and it is felt that the pulling test will be more acceptable approach on load transfer capability determination as it reflects on the realities of bolting application and that the bolts installed as a supporting element undergoes tensioning.

5 PUSH TEST

Figure 6a shows a typical short encapsulation test sample with the tested bolt being installed in 75 mm long steel cylinders. In hole diameter is 27 mm. Figure 6b is general arrangement for push testing. The lining of the 27 mm hole cylinder was grooved to provides grip for the encapsulation medium and prevents premature failure on the cylinder/resin interface. Further details of the steel sleeve can be found in Aziz and Webb (2003 a) and Aziz and Webb (2003 b). The rock bolt samples tested were each cut to 120 mm in length. The equal lengths bolts ensured that all the samples of the same type had an equivalent number of profile ribs and that the ends of each sample were square. All bolts were encapsulated into the push test cells using Minova PB1 Mix and Pour resin grout of uniaxial compressive strength and shear strengths in the order of 70 MPa and 16 MPa respectively.

5.1 Results and discussion

Table 2 shows the details of test results for various bolts. Figure 7 shows typical loaddisplacement graphs of testing Type T2 bolts. The results of four tests are shown, and demonstrates the repeatability of the tests



Figure 6a. Short encapsulation unit.

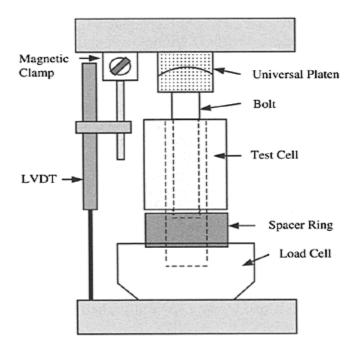


Figure 6b. Instrumented push tests arrangement.

with a reasonable degree of confidence. Figure 8 shows the combined load displacement graphs of a group of four popular profiled bolts. Clearly, there are differences in the graphs of different bolts and one notable example is that of Bolt Type σ 3. This bolt had widely spaced profiles, and the peak load occurred at greater displacement than the rest of the bolts. Table 2 shows the details of the test results for all profiled and plain surfaced bolts. These results are the average values for the maximum load, shear strength, and bolt

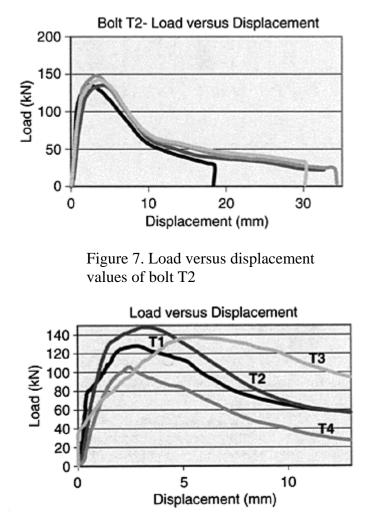


Figure 8. Load displacement profiles of four popular profiled bolts.

	Sample	e type						
	Popular Additional							
Bolt type	T1	T2	T3	T4	T5	T6	S1 Smooth	S2 Rough
Ave profile height (mm)	0.70	1.40	1.20	0.70	1.24	1.12	_	_
Ave profile spacing (mm)	11.00	12.00	25.00	11.00	12.00	3.00	-	-
Ave max load (kN)	117.12	132.56	115.11	102.09	121.32	112.79	43.59	113.64
Ave max displacement (mm)	2.87	3.31	5.62	2.57	2.09	2.37	0.57	1.01
Ave shear stress capacity (MPa)	22.88	25.89	22.33	19.88	25.00	21.79	8.58	22.35
Average system stiffness (kN/mm)	40.72	40.05	20.48	39.72	57.94	47.59	76.47	112.40

Table 2. Push test characteristics of different bolts values—Average values.

resin interface stiffness values. Figures 9 and 10 shows the peak load and displacement bar charts respectively.

Clearly the average displacement results of Bolt Type T3 achieved the greatest displacement and concur with the findings obtained from CNS tests as discussed earlier. Figure 11 shows the average shear

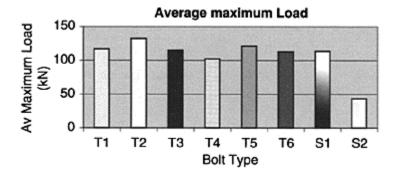
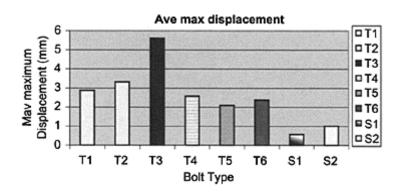
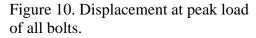
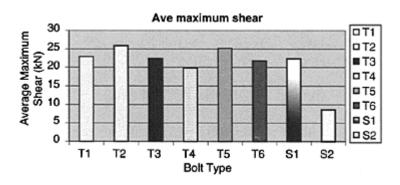
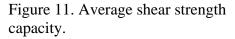


Figure 9. Average peak load of all the bolts.









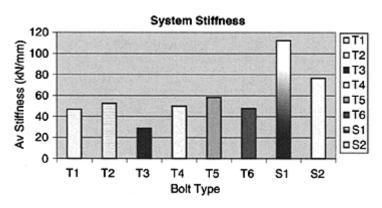


Figure 12. Average bolt anchorage system stiffness.

strength capacities achieved by each bolt type. It was found that Bolt Type T2 had the highest shear strength capacity of 25.89 MPa.

The lowest shear strength value of the popular bolt type was Bolt Type T4 at 19.88 MPa, which was 23.21% less then the shear strength value of Bolt Type T2.

The rough surfaced plain bolt achieved a shear strength capacity of 22.35 MPa, and the smooth plain surface bolt achieved 7.71 MPa, which was a large drop in the shear strength values with respect to rough surfaced plain bolt.

Testing of Bolt Types T3 and T1 were used to examine the effect of profile height on the shear strength capacity across the bolt resin interface. Bolt Types T1 and T3 were of the same "T" bolt design, possessing similar profile spacings, but had different profile heights. As outlined in Table 1, Bolt Type σ 3 had a profile height of 1.4 mm, while Bolt Type T1 had a height of 0.8 mm. However, both Bolt Types T3 and T1 achieved shear strength capacities of 25.89 MPa and 22.88 MPa respectively. Bolt Type T2 achieved a greater shear strength capacity compared to Bolt Type T1. Figure 12 shows the average system stiffness of various bolts, which is also shown in table 2. The system stiffness is the gradient of the maximum load sustained by a bolt to the displacement of a fully encapsulated bolt. It is interesting to note that both smooth surfaced bolts were stiffer than the profiled bolts, however this does not mean that the plain surfaced bolts have greater load transfer capacity, as the displacement at peak load was very minimal. Also and as shown in Figure 9 the increase in surface roughness of the plain surface bolt has greatly influenced the shear strength capacity of the bolt. The rough finish of the bolt surface allowed additional grip to be provided between the bolt and resin interface and this reinforces the belief that rusted bolts have greater load transfer capability than a clean bolt of the same type.

6 PULL TEST

Figure 13 shows the test set-up for pulling a bolt out of 75 mm steel sleeve, and Figure 14 shows the pull test load/displacement results of Bolt Types T1 and T2.

Table 3 shows the comparative test results of pull and push tests. Both push and pull test samples were prepared from the same premix resin batch. Also included in the table are the average values of push tests.

There were some variations in the values for different bolts. The difference between the average push and pull test results for both Bolt Types T1 and T2 were in the range of between 8 and 11% respectively. Further research is continuing to examine other profiled bolts.

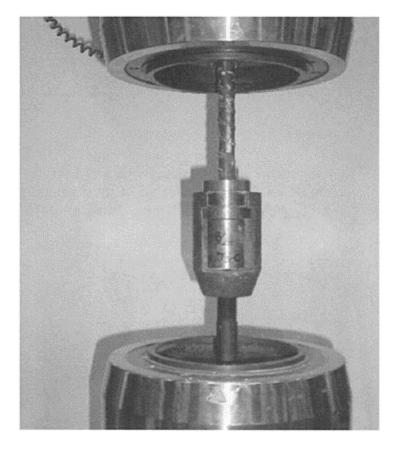


Figure 13. Pull tests arrangement.

7 CONVENTIONAL PULL TESTS LABORATORY AND FIELD STUDIES

7.1 Laboratory test

The laboratory experimental work was carried out in a purpose built testing rig facility pictured in Figure 15a. The rig consisted of a double deck steel frame structure. The upper deck carried a drilling medium of a block of rock or concrete and an overhead-lifting crane (not shown in the figure) used for lifting and placement of the drilled medium. A hydraulic drilling rig, positioned beneath the drilled concrete block, was adapted from a continuous miner as reported by Aziz (2004).

The high strength concrete block had a 1.0 m^2 base area that tapered to 0.9 m^2 area at the top, and an overall height of 1.2 m. Figure 15b shows the general arrangement for pull testing of the bolts. The hydraulic ram had a maximum capability of 30 tonnes.

A total of 55 bolts were installed in three different borehole diameters of 27, 28 and 35 mm respectively. The tested bolts were Bolt Types T1, T2 and T3. 45 bolts were installed in the concrete block using resin cartridge and the remaining 10 bolts were installed with PREMIX resin (known as Mix and Pore 'P1' Resin). Premixing involved mixing the resin in a container

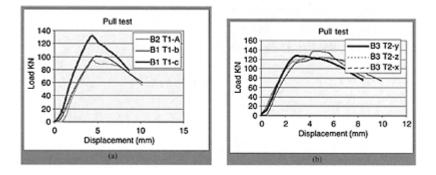


Figure 14. Pull test results for Bolt Types T1 (a) and T2 (b).

Table 3. Data from push and pull tests.

Bolt type	Av peak push load and SD (kN)	Av peak pull load and SD (kN)	Diff (%)	Ave peak load displacement and SD in push test (mm)	Ave peak load displacement and SD in pull test (mm)
T1	130.7 (±10.85)	120.06 (±16.4)	8	3.55 (±0.63)	4.60 ± 0.32
T2	140.31 (±6.0)	129.4 (±6.95)	11	4.3 (±0.82)	4.2 ± 0.73

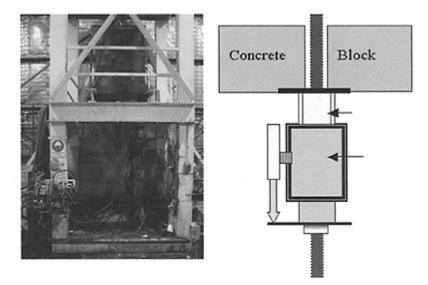
and pouring it into the hole around the bolt in the inverted concrete block.

7.2 Field test

Field tests were carried out at the intake side of an underground local coalmine in the Illawarra Coalfield of NSW, Australia. All the holes were drilled in medium to coarse sandstone, which can be described as a competent formation. Three hole sizes were used with anchorage lengths being maintained at 300 mm. The holes were initially drilled at 500 mm in length and the first 200 mm length was then reamed to 35 mm. A total of 36 bolts were installed in three different bolt diameters of 27, 28 and 35 mm respectively.

8 RESULTS AND DISCUSSIONS

Table 4 shows the average peak loads and peak displacement of all three bolts tested in the laboratory and in the field using different diameter holes.



a) Drill Rig

b) Bolt Pulling System

Figure 15. Laboratory drill rig and bolt pull test arrangement.

Table 4.	Summary	of average	results.

		Average j (kN)	peak lo	ad	Displace peak loa		Average stress (N		Shear stiffness KN/mm
Bolt	Hole diam eter (mm)	Lab	Field	Lab	Field	Lab	Field	Lab	Field
T1	27	246	190	8.05	25.1	13.9	9.30	50.0	35.0
T1	28	167	154	5.75	9.4	9.5	7.53	47.0	20.0
T1	32	>300*/66	75	3.54	8.9	2.8	3.67	_	-
T2	27	251.7	229	6.29	14.5	14.2	11.19	46.0	53.0
T2	28	235.8	155	7.04	8.0	13.3	7.58	38.0	22.0
T2	35/32	>300*	68	_	_	>16.9*	3.3	_	-

T3	27	>300	251	15.56	42	17	12.27 53.0	47.0
Т3	28	252.8	179	12.63	12	14.3	8.75 46.0	17
T3	32	>300	16	_	3.0	>16.9*	0.78 –	-

NB: * Premix resin encapsulation.

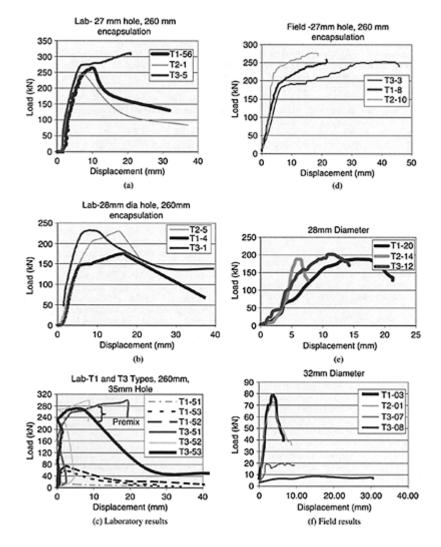


Figure 16. Laboratory and field testing results for different bolts.

Figures 16 a, b and c show the laboratory results of the pull tests carried out on three different bolts and in three different diameter holes. Figures 16 d, e, and f show the field test results of the peak load and displacement values of similar type bolts in three different borehole diameter holes. Clearly the methodology of resin encapsulation application had some influence on bolt anchorage performance. The pull out anchorage loads for premix encapsulation far exceeded those obtained from cartridge types irrespective of bolt type and annulus thickness.

The laboratory load displacement results shown in Figure 16 were obtained from 260 mm long encapsulation instead of the conventional length of 300 mm. It is obviously clear from Table 4 that the pulling forces in the laboratory situation at 300 mm length anchorage was in-excess of the Yield strength of the all the bolts tested. However, such situation was less of an occurrence in the filed were 300 mm long resin encapsulation was used. The results from two different testing conditions have produced near similar trends. As expected, the peak pull force for widely spaced profiled Bolt Type T3 occurred at greater displacement than both Bolt Types T1 and T2 respectively as indicated in Table 2. Such behaviour is similar to that obtained from both the CNS test (Aziz and Dey, 1999) and short encapsulation test (Aziz and Webb 2003 a, and 2003 b). In particular, the displacement at peak load was greatest for Bolt Type T3. This was followed by Bolt Type T2 and the least displacement was for Bolt Type T1.

The following can be deduced from both the laboratory and field test results as listed in Table 2 above:

- 1. The peak load displacement varied according to the bolt profile configuration. There was very little difference in displacement at peak load between two equally spaced Bolt Types T1 and T2 profiles, however the displacement was greater in widely spaced Bolt Type T3.
- 2. The above point (1) finding was in agreement with previous reporting by Aziz (2002) Aziz and Webb (2003 a), and Aziz and Webb (2003 b).
- 3. For all three bolt Types, the average peak pulling force values and displacement at peak load was highest in the 27 mm diameter holes. This was followed by the 28 mm holes and with the least values being obtained in 32/3 5 mm holes. However, the variation in peak loads with respect to borehole diameter did not hold when the bolts were anchored with pre-mix resins encapsulation.
- 4. Premix resin encapsulation was found to be superior in performance to the cartridge type. This is obviously clear from the results of the tests in the laboratory for all three bolts and as evident Figure 16c.
- 5. The reduced performance of pull out force with increased annulus thickness was considered to be attributed partly to insufficient resin mixing leading to excessive gloving.

9 CONCLUSIONS

1. CNS method of evaluating the influence of different bolt profile configuration is a realistic method of determining the load transfer capability of the bolt.

- 2. The methodology of removing the bolt in short encapsulation tests has an influence on the pulling results. There was an average difference of 10% between the bolts pushed and pulled out of short encapsulation tests.
- 3. Rib profile height influenced the shear strength capacity of a bolt.
- 4. The rough finish of the bolt surface permits additional grip between the bolt and resin interface and this enforces the belief that rusted bolt surfaces have greater load transfer capability than clean surface bolt.
- 5. Premix resin encapsulation was found to be superior in performance to the cartridge type resin.

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Extent and mechanisms of gloving and unmixed resin in fully encapsulated roof bolts and a review of recent developments

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ABSTRACT: Effective strata control, utilising fully encapsulated roof bolts is dependent on the installed quality of the reinforcement elements. One mechanism by which roof bolts may become less than fully efficient is by glove fingering (gloving) and insufficient or non-mixing of the resin (unmixed). Following a routine installed bolt quality audit and some small roof failures containing gloved bolts, a work programme was initiated to determine the extent of gloving and the unmixed resin problem and to develop an understanding of mechanisms involved. Results have shown that gloving and unmixed resin is a systematic and widespread phenomenon, occurring across the range of resin and/or bolt manufacturers, and in a variety of roof types. Gloving was found in bolts installed using either hand held pneumatic or continuous miner-mounted hydraulic bolting rigs, under typical mining conditions, and under controlled manufacturers "best practice" conditions.

The mechanisms involved have been confirmed as being the development of a pressure front as the bolt encounters the resin cartridge and is spun up the hole, which in turn, leads to over-pressurisation and radial expansion of the resin cartridge. The result is an increase in the diameter of the plastic cartridge. This allows the bolt to be spun up inside the cartridge without making sufficient contact to shred the cartridge or the hardener envelope, typically resulting in a portion of the cartridge enveloping the bolt and containing unmixed resin mastic and catalyst. Once the mechanisms involved and extent of the problem became clear, further research was undertaken to assess alternative bolt and resin cartridge profiles and modifications in an effort to minimise and/or eliminate the gloving and unmixed resin phenomenon. Research has been undertaken using recovered bolts from various mine sites, as well as test bench trials and the quantification of the loading characteristics of gloved bolts using strain-gauged roof bolts.

To understand the impacts of gloved bolts and unmixed resin on roof control, failure pathways and reinforcement requirements, a FLAC 2D

numerical simulation was undertaken. The results were incorporated into the strata management plans for the affected operations. Laboratory data has been collected and analysed to assess the magnitudes of resin pressure as the bolt encounters the cartridge. Data was also collected on the affects of gloving and unmixed resin on the load transfer characteristics of the resin-bolt system.

1 INTRODUCTION

Gloving, in this context, refers to the plastic cartridge of a resin capsule partially, or completely, encasing a length of bolt, typically with a combination of mixed and unmixed resin filler and catalyst remaining within the cartridge. The gloved and unmixed portion reduces the effective anchor length and adversely affects the ability to reinforce the roof strata. Figures 1, 2a and 2b illustrate typical examples of gloved roof bolts and unmixed resin/catalyst.

Gloving has been recognised in fully encapsulated roof bolts for many years (Pettibone, 1987) and has traditionally been attributed to poor installation methods, issues relating to drilling/installation equipment, poor handling and/or storage of consumables or geological issues resulting in abnormal ground conditions. Very little quantitative data is available in the public domain, although recent work by Campoli *et al.* (2002) and Campoli, Mills, and Adams (2003), Fiscor (2002), Campbell and Mould (2003), and Pasters (2003), along with hazard alerts from the Queensland Mines Inspectorate (Qld Gov't 2002) have brought some focus to the problem.

Fiscor (2002) refers to gloving being identified as the cause of roof falls in American operations; in one of

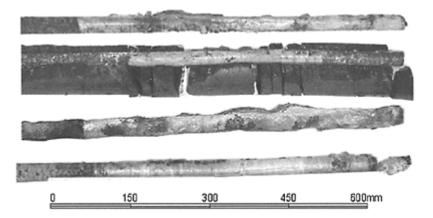


Figure 1. Typical appearance of gloved bolts and unmixed resin.



Figure 2(a). Upper photo shows intact core in stone roof. Lower photo illustrates how once the core is removed the gloving and unmixed resin is revealed, in this case 550 mm was found to be gloved and unmixed. Note: Bolts are photographed with the top of the bolt on the left hand side.



Figure 2(b). Typical examples of gloved bolts and unmixed resin. Note: Bolts are photographed with the top of the bold on the left hand side.

the few published references directly linking gloved bolts and falls of ground.

2 LOAD GENERATION AND TRANSFER IN RESIN GROUTED ROOF BOLTS

The performance of any reinforcement design is limited by the efficiency of load transfer of the reinforcing members. Load transfer is the mechanism by which force is generated and sustained in the roof bolt as a consequence of strata deformation (Fabjanczyk and Tarrant, 1992).

In a fully grouted roof bolt the load transfer mechanism is dependant on the shear stress developed on the bolt-resin and resin-rock interfaces. The peak shear stress capability of the interfaces and the rate of shear stress generation, determines the response of the

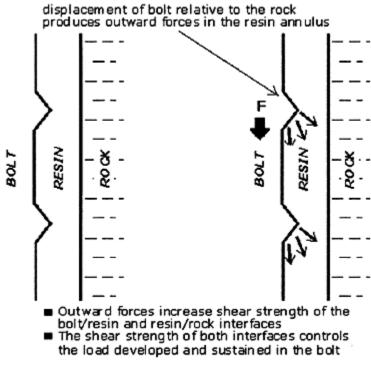


Figure 3. Load transfer in a roof bolt (after Fabj janczyk and Tarrant 1992).

bolts to the strata behaviour. This concept is illustrated in Figure 3, where shear stress is generated in the resin between the bolt and rock.

3 OVERVIEW OF INVESTIGATION PROGRAM

The work program reported here was undertaken over the past 2 years as part of an audit of the installed quality and effectiveness of roof bolts in various Solid Energy NZ Ltd. coalmines. The initial investigation program concentrated on defining the extent of the problem and developing an understanding of the mechanisms involved. This was then followed by a series of trials aimed at finding an acceptable solution. The later involved trials of alternative bolt/resin suppliers and modifications made to bolts and bolt ends based on the understanding of the mechanisms involved.

The overcoring method allowed the recovery of "typical" bolts installed by operators at the face and in the back-bye districts of the mines, as well as to audit bolt installations under controlled manufacturers "best practice" conditions using either pneumatic (gopher/wombat) or hydraulic (miner mounted) drilling rigs. At each operation standard drilling consumables were used, and a strict measurement and recording protocol was defined to ensure the same parameters were recorded so that all results were comparable. Overcoring was undertaken using specialist techniques and equipment, drill bits and barrels developed by SCT Operations, using either a Proram or Ramtrak drill rig. Stone roof was typically drilled with a diamond bit and coal roof with a tungsten-tipped bit. In addition to the overcoring program, bolts of various type, age and manufacturer have been collected from falls of ground and goaf edges for comparison and to assist in gauging the extent and history of the issue. Further quantitative data has also been gathered using strain-gauged bolts, installed to measure the loading characteristics of the gloved bolts.

4 EXTENT OF THE GLOVING PROBLEM

In excess of 90, 1800 mm long roof bolts of four different types/manufacturers were overcored and recovered in the initial investigation. The bolts were recovered from five separate operations, in different geological and geotechnical settings, with the immediate bolted section comprising a range of lithologies from coals to shale/mudstones and sandstone/laminites. The overcored bolts investigated had been installed for time spans, ranging from hours to 18 months.

Bolts and resins used in this program are from various manufacturers and suppliers in Australasia. The resin cartridges used were all two-part polyester (fast/ slow setting times), 900–1000 mm long and nominally 24 mm in diameter. The bolts used were all flat topped, with a core thickness of 22 mm and 1 to 2 mm ridge profiles, with ridge spacing depending on manufacturer and bolt type.

The investigation program was undertaken in two phases, the first being the recovery of "typical" mineworker installed bolts from throughout the five mine sites to assess if gloving was routinely occurring. The second phase was to undertake controlled "best practice" installations (often with a manufacturers representative present) to determine if an acceptable quality bolt (no gloving and fully mixed) was achievable. Both data sets are presented here. The results of the investigation work are summarised in Figure 4 in terms of bolt and resin type/manufacturer, roof lithology and installation type, ie controlled or "typical".

As illustrated in Figure 4, regardless of bolt type, roof lithology or if the bolts were installed by mineworkers or under best practice controlled conditions, an average of 500 mm of bolt length is typically affected by gloving, with 200 mm typically gloved and mixed and 300 mm typically gloved and unmixed. Although it must be noted that there is a wide range in values in the data set, (from 30 mm to 790 mm), with the majority of the data set (70%) having in excess of 400 mm of unmixed resin (nearly exclusively at the top end of the bolt).

The overcoring program graphically illustrated that gloved bolts and unmixed resin were common at all sites investigated regardless of roof lithology and occurs across the range of bolt and/or resin manufacturers tested. The results also indicate that there is little difference in "typically" installed bolts and those installed under "best practice" controlled conditions.

Extent and mechanisms of gloving and unmixed resin 421

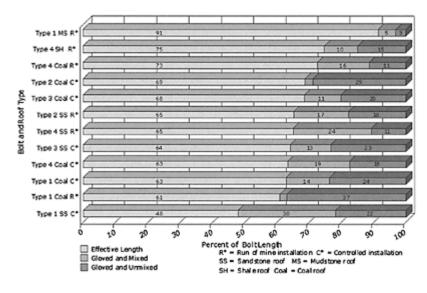


Figure 4. Summary of results for investigation into the extent of gloving of standard bolts. (Note: each type represents a set of bolts and is displayed as the "mean" lengths for the set of bolts).

In fact, it is apparent that an acceptable quality bolt is typically not achievable using the current flat-topped bolt and cartridge systems on the market. As such it can be concluded that gloved bolts and unmixed resin can not be solely attributed to poor installation methods, issues relating to drilling/installation equipment, poor handling and/or storage of consumables or geological issues resulting in abnormal ground conditions. Rather it is predominantly be a bolt and resin system problem.

In addition to the overcored bolts, several areas of falls in back-bye regions and goaf edges of mines were inspected in an attempt to find bolts of differing ages and resin cartridge configuration to determine the time span over which gloving has been a problem. To date, bolts ranging in age from hours to in-excess of 12 years have been recovered, all of which show a similar degree of gloving and/or unmixed resin, indicating that it is an issue that has been prevalent in the industry for a considerable time.

5 MECHANISM FOR GLOVING AND UNMIXED RESIN

Determining the mechanism for gloving and the resultant unmixed resin has been the focus of much of the preliminary investigation work. From observations of bolt installation, overcored bolts and from test bench trials and measurements, an

understanding of the mechanisms has been developed and confirmed. This is discussed below.

A pressure front develops when the standard flattopped bolt encounters the resin cartridge as it is spun and pushed up the hole. The bolt acts essentially as a piston hydraulically pressurising the resin and cartridge, which shortens and in turn undergoes radial expansion until it is confined by the sides and the back of the drill hole. The result being, an increase in the cartridge diameter from 24 mm to the diameter of the drill hole, normally 27 to 29 mm. In conjunction with the expansion against the drill hole wall, the catalyst envelope/tube also becomes flattened against the side of the drill hole. The increased cartridge diameter allows the bolt (nominally 22 mm core and 24 mm rib diameter) to be spun up inside the cartridge without making sufficient contact to shred the cartridge or the flattened hardener tube. This culminates in a portion of the bolt (typically the top end) encased in intact cartridge and typically a combination of mixed and unmixed resin mastic and catalyst.

To confirm the causes of gloving, test bench simulations were undertaken in clear 28.5 mm internal diameter polycarbonate tubing so that direct observation of the gloving mechanics could be made as the bolt

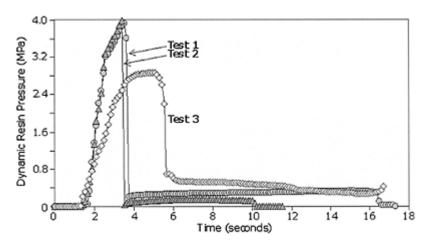


Figure 5. Measured up-hole dynamic resin pressure as bolts are spun through a resin cartridge (after Mills, 1999).

was installed. Visual conformation of the pressure front development and of the bolt entering the cartridge (gloving) was possible.

The observations showed that the pressurisation of the cartridge reaches sufficient levels to expand the cartridge against the wall of the tube at 600–700 mm from the end of the 1800 mm tube, and it is at this point the bolt enters the resin cartridge. Once the bolt was spun up inside the expanded cartridge no further shredding of the cartridge took place, resulting in a 600–700 mm gloved and unmixed resin portion of the bolt, replicating the field observations, and confirming the mechanisms involved.

6 UP-HOLE DYNAMIC RESIN PRESSURE

Up-hole dynamic resin pressures have previously been measured (Mills, 1999) using a pressure transducer through a hole in the top cap of a 27–27.5 mm polycarbonate tube under test bench conditions designed to closely simulate field installation of roof bolts. The dynamic pressure was shown to rise to its peak of around 4 MPa very rapidly as the bolt was spun up the hole with the maximum pressure developed after 4 seconds, which equates to the bolt being approximately 1100 to 1200 mm up the hole. Figure 5 illustrates the results generated.

It is worth noting that the peak values recorded may not be the actual maximum pressure due to problems maintaining the transducer fittings. Pressures were such that the end-cap detached and in some cases the polycarbonate tube split.

Up-hole resin pressures in excess of 4 MPa are considered to be sufficient in magnitude to induce hydraulic-fracturing of strata and pre-existing planes of weakness in situations where minor horizontal stress is of the same order of magnitude. Hydraulic fracturing causing the initiation or opening of joints or cleat, results in the injection of resin into the strata and can lead to significant loss of encapsulation length. Indeed resin injection into strata was commonly observed in overcored bolts along with considerable loss of encapsulation. Typically resin could be observed in the drill core to radiate out from the bolthole to excess of the diameter of the recovered core (Ø100 mm) and be in veins up to 2–3 mm thick. Examples of resin injection as a result of hydro fracturing are illustrated in Figures 6a, 6b and 6c. The volume of resin loss is sufficient to significantly reduce the encapsulation length, which was found in extreme cases to be less than the length of the installed resin cartridge.

7 TRIALS OF ALTERNATIVE BOLT PROFILES AND TIP CONFIGURATIONS

Following the recognition of the extent of the problem and the understanding of the mechanisms involved, a considerable effort was made to develop a solution, using bolt tip modifications suggested by a range of sources (operators, manufacturers, consultants) and

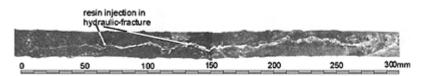


Figure 6(a). Example of resin injection into an induced fracture.



Figure 6(b). Resin injection into an induced fracture in coal.



Figure 6(c). Resin injection along a pre-existing joint in an inter-bedded sandstone and mudstone roof.

alternative profiles available on the market as recommended by various bolt manufactures.

Trials were undertaken across several of the mine sites to cover the same range of geotechnical conditions. In total a trial of 9 alternative profiles and tip modifications was undertaken, with details of bolts given below in Table 1. Some 95 bolts were recovered across the spectrum of modifications and mine sites. At all trial sites standard bolts were also installed and recovered as a control to ensure that the modifications could be directly compared to standard profile bolt types.

The results of the trial are summarised in Figure 7, which illustrates the average proportions of bolt length that were affected by gloving and unmixed resin. Initial trials (June 2002) were carried out using the chamfered, horn, welded, paddle, peeled and peeled and threaded bolts. The spiralled wire, 200 mm wiggled and off-centre type of profiles were tested in May 2003.

It can be clearly seen that of the modifications tested only the chamfered, wiggled and off-centre bolts offered any significant improvement, with each achieving in excess of 90% effective bolt length. The remaining modifications showed only minor or very inconsistent improvements in both mixing and shredding of the cartridge. The result of the modification and alternative bolt profile trial indicates that significant improvements in resin mixing and shredding of the cartridge was consistently achieved using the chamfered, wiggled and off-centre types of bolts.

The results of the initial trial of the chamfered bolt were considered so successful at the time that this modification was adopted as the standard bolt profile in a large-scale trial at one of the mine sites involved. However, the chamfer does have a tendency to puncture the resin cartridge very easily and consequently gloves readily, particularly in a low roof mine (2.5 m) and where longer bolts (1.8 m & 2.4 m) are used. More recently

the 200 mm wiggled profile bolt has been introduced as the standard roof bolt at that mine.

Testing of a plastic mixing or anti-gloving device, which is fitted to the up-hole end of the bolt, has been tested. The anti-gloving device has a sharp upper profile which ruptures the resin cartridge, grabs the cartridge sleeve and twists the sleeve to the back of the hole as the bolt is spun up the hole. Grooves in the anti-gloving device promote flow of resin past the device and bolt as it is spun up the hole.

Results from the five bolts that have been overcored have been encouraging (Figure 8). The two bolts that showed signs of gloving were over-drilled

Modification Type & Description	Schematic Diagram
Chamfer Curved wedge of bolt removed from tip	27mm
Welded Bar of weld built up on opposite sides of bolt	27mm
Horn Two 'horns' built-up onto tip and sides of bolt	27mm
<u>Paddled</u> Tip flattened to form a paddle shape	\bigcirc
Peeled & Threaded Ribs peeled off and thread 160 mm of bolt, rolled on	
Peeled Ribs peeled off 160 mm of bolt	
Spiralled Wire attached to tip Spring wire and wound along bolt	
Wiggle Standard bolt with wiggle profile over upper 200 mm of bolt	Contrastor and

Table 1. Details of modified and trial bolt profiles.

Off Centre Nut made off centre relative to the bolts long axis	22
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which is likely to have contributed to their gloving. Further overcoring is planned in order to obtain a more statistically meaningful data set.

Further tests are planned with resin cartridges sourced from outside Australasia. These cartridges are constructed with a larger cross sectional area for the catalyst tube. Discussions have been held with various resin manufacturers about constructing various cross sectional areas and profiles for testing.

8 LOADING CHARACTERISTICS OF GLOVED BOLTS AND UNMIXED RESIN

The use of instrumented roof bolts to quantify the loading characteristics of a gloved roof bolt was investigated. The results presented in Figure 9 show that the bolt (which was installed as a "typical" bolt by operators) was indeed gloved over the upper 400 mm, as indicated by nil load transfer over that portion of the bolt length. The resultant loading profile illustrated that the bolt was able to generate significant load over the shorter length and was not approaching yield.

The use of a modified instrumented bolt as part of a "typical" installed bolt quality audit is currently being explored, as the results shown in Figure 9 illustrate that they are able to identify the extent of gloving and may be a cost effective, non-destructive alternative to overcoring as a means of assessing if gloving and unmixed resin is occurring in an operation.

9 LABORATORY PULL TESTS TO QUANTIFY THE EFFECTS OF GLOVED BOLTS AND MIXED RESIN

Gloving of the bolt by the cartridge, where the resin has hardened, may adversely affect the load transfer characteristics of the bolt system by reducing the system stiffness and by providing a low friction interface reducing the magnitude of the shear stress sustainable on the bolt/resin/rock interfaces.

Short encapsulation pull tests undertaken by the United States Bureau of Mines (USBM) on gloved bolts indicated that there was no detrimental affect to the load transfer characteristics caused by mixed and hardened resin encased in the plastic resin cartridge (Pettibone, 1987). Pettibone also reported that the plastic cartridge did not provide a surface, which promoted or allowed shear along it under heavy loads. The detrimental effect of gloving on short encapsulation pull tests has been discussed (Mazzoni et al., 1996; Mark et al, 2002).

In order to attempt to quantify the impact of gloved and mixed resin on load transfer characteristics laboratory short encapsulation pull tests (180 mm) were undertaken at the University of New South Wales School of Mining using the rock bolt testing facility

(pers. comm). Inspection and analysis of short encapsulation pull test specimens was undertaken to quantify relative proportions of gloving/unmixed and gloving/ mixed. All of the test specimens exhibited some degree of gloving and unmixed resin, with only 4 specimens having significant (greater than 10 mm) gloving and mixed. The peak shear strengths (normalised for the actual effective resin length) were then correlated to the proportions of gloving and mixing.

The results, while not considered to be statistically representative, do indicate that the load transfer of the bolt/resin system is not significantly reduced by a gloved and mixed section of bolt, with the data scatter being within the same range of results as that of the shredded and mixed, full length bolts.

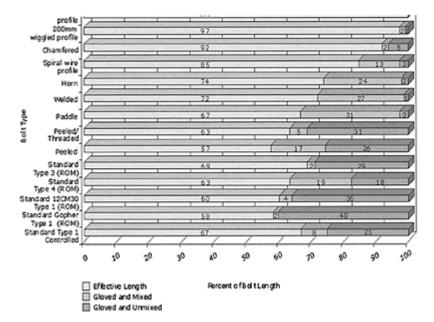


Figure 7. Summary graph of average modified bolt performance. (Note: Further testing of the off-centre profile has revealed that the bolt is not 100% effective. Further testing is underway on the off centred bolt to build a more statistically meaningful data set).

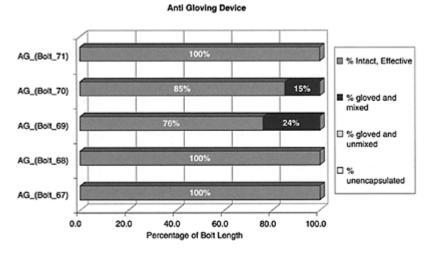


Figure 8. Overcoring results from bolts capped with anti-gloving device.

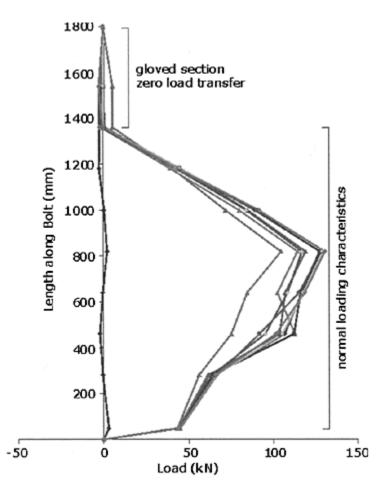


Figure 9. Result of an instrumented roof bolt showing nil load transfer over gloved section.

10 IMPLICATIONS FOR DESIGN

The wide spread nature of the gloving problem led to the realisation that the current roof support design assumptions were compromised along with the subsequent deformation trigger levels which form a major component of most strata management plans. The decision was made to case study a mine site where a calibrated numerical (FLAC 2D) model existed and a significant amount of field observation and roof deformation monitoring data was available to assess the impacts of the reduction in effective bolt length on the strata behaviour and stability of an opening.

The FLAC model was used to simulate two different support configurations using 1800 mm bolts. In the first case it was assumed that the bolts were fully encapsulated, and 100% effective. In the second the top 600 mm of the bolt was gloved and encased in unmixed resin, effectively making the bolts 1200 mm long. The gloved and unmixed section will still provide shear resistance, however, unmixed resin was not factored into the model. The length of gloving and un-mixing was consistent with the recovered bolts for that operation.

All other parameters were kept constant in the models, these being the excavation size/geometry, the geology and geotechnical setting (depth of 200 m, 4 m

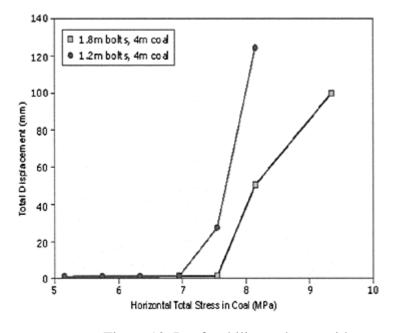


Figure 10. Roof stability pathway with respect to applied horizontal stress.

coal roof) and load transfer characteristics of the effective bolt lengths.

A range of horizontal stress magnitudes (5–10 MPa total stress, based on previous 3D over core stress measurements) were simulated to compare against the performance of 1.2 m and 1.8 m long bolts in the same environment. In this way the general stability curve (with respect to overstressing) for each bolt length is established.

Figure 10 details stability curves for the 1.2 m and 1.8 m long bolts showing total roof displacement versus horizontal stress. The modelling indicates that the gloved and unmixed 1200 mm bolts became overloaded earlier (at a lower stress level) than the 1800 mm long, 100% effective bolts.

In the case of the 1200 mm bolts, once over stressing and shear initiates in the roof, the failure path is rapid, with softening above the bolts and mobilisation of the contact at the top of the seam at lower displacement levels. For example, softening above the bolts,

occurs at a displacement of 10 mm for 1200 mm bolts, compared with 15 mm for 1800 mm long bolts.

Figures 11a and 11b are FLAC model outputs of strata softening occurring for the same horizontal stress level. The magnitude of the horizontal stress modelled is 7.5 MPa (total stress) in both situations, which was considered representative of the mining conditions at the time, based on observations of guttering and roof deformation levels. It can be seen that the longer bolts are better able to confine the immediate roof and act as a pattern to cope with a wider range of conditions while the gloved bolts act more as isolated reinforcing members and allows greater shearing and mobilization of the coal roof.

From the modelling results the strata trigger levels have since been revised and incorporated into an

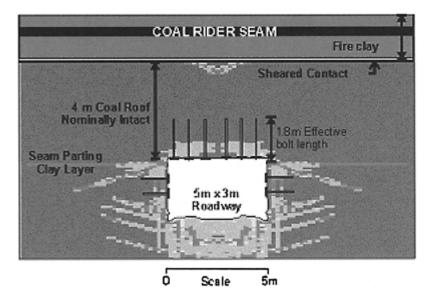


Figure 11(a). Results of the FLAC modelling of 1800 mm long 100% effective bolts. (Note: Coal roof and parting modeled as discontinuities).

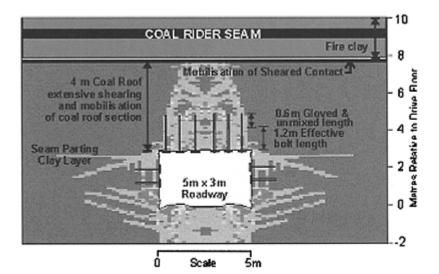


Figure 11(b). Results of the FLAC modelling of 1200 mm effective length, gloved roof bolts. (Note: Coal roof and parting modeled as discontinuities).

updated Strata Management Plan for the affected areas. Simultaneously a review and reanalysis of the existing roof deformation monitoring data (from wire extensometers) was undertaken to assess which areas of the mine were potentially requiring remedial secondary support at the revised lower magnitudes of deformation.

11 SUMMARY AND CONCLUSIONS

Investigation work has identified gloving and unmixed resin in resin-grouted roof bolts as a significant problem, which occurs across a range of geological/ geotechnical settings, and across the range of bolt/ resin manufacturers.

Gloving was found over a wide range of bolt lengths, with the results showing anywhere between 30 mm to 790 mm affected. Typically the gloving affected around 500 mm of the up-hole end of the bolt. The length of bolt affected by unmixed resin also varied, with up to 750 mm of bolt encased in soft, unmixed resin.

While the affects of gloved but mixed resin may fbe minimal, the unmixed portion affords no axial reinforcement to the roof. The impacts of this can be assessed on two fronts, the first being with respect to health and safety, and the second being the economic cost. In terms of primary reinforcement dollars, this equates to 10% to 30% of the reinforcement dollars being of no benefit.

A mechanism, which accounts for the gloving and unmixed resin phenomenon has been described, and validated by field and test bench trials, and measurements.

The over-pressurisation of the resin column as the bolt is spun up the hole results in the radial expansion of the cartridge and flattening of the hardener tube against the borehole wall. The bolt enters the expanded cartridge and does not shred the hardener tube, often resulting in a gloved section of bolt and/or unmixed resin.

Nine bolt modifications and alternatives were tested across a range of geotechnical conditions, with the best results being achieved by the off-centre, Chamfer and a 200 mm wiggled bolt, with the latter being introduced as the standard bolt profile at one operation.

An anti-gloving device attached to the up-hole end of a bolt has shown encouraging potential to reduce the occurrence of gloving and unmixed resin. Resin cartridges with a larger catalyst tube cross sectional area are currently being tested.

Numerical modelling was used to assess the impacts of gloving and unmixed resin and showed that the shorter effective bolt length does have a significant affect on the design assumptions and stability of mine openings. In the case presented, the shorter bolts could not interact as effectively as a pattern and the reinforcement afforded to the roof was reduced, compared to the design assumption of 1800 mm long bolts. The result being: an increased height of softening at lower levels of deformation, leading to the isolation and over stressing of the immediate and secondary roof sections. Following the results from the modelling a review of the Strata Management Plan was required to incorporate the findings, and a review of all monitoring data was required to reassess the requirements for secondary support.

Strain-gauged bolts have been used to assess the loading characteristics of the gloved bolts and unmixed resin, and may provide a means of assessing gloving of "typical", miner installed bolts, on a regular basis as part of an audit process.

ACKNOWLEDGEMENTS

The authors would like to take the opportunity to thank Solid Energy New Zealand Ltd for instigating and supporting this research program. The support and enthusiasm from all levels of Solid Energy NZ Ltd proved invaluable, along with the provision of logistical support, funding and for providing invaluable access to their operations during the overcoring programs. Finally the authors would like to acknowledge Solid Energy NZ Ltd and the bolt and resin manufacturers (including Celtite Pty Ltd) for allowing the publication of the data sets.

*R.N.Campbell worked for Solid Energy NZ Ltd when this research commenced.

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The effect of rock strength on shear behaviour of fully grouted bolts

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ABSTRACT: Load transfer mechanism of fully grouted rock bolt is influenced by a number of parameters. These parameters include; medium properties, pretension effect, bolt profiles, steel type and bolt and hole diameters. The effect of enclosed medium/rock strength on the rate of load transfer capacity is studied. Laboratory tests consisting of double shearing of bolts in three-piece concrete blocks were conducted to examine the influence of bolting on the shear strength of the joints in the host medium. Tests were conducted at different bolt pretension loads using different bolt profile configurations. A three dimensional finite element method (FEM) was used to simulate the laboratory test shearing process. The study showed that there was a direct relationship between host medium (rock) strength, and the resistance to shearing force and shear displacement.

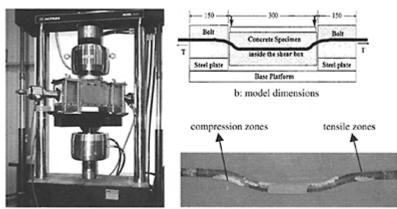
1 INTRODUCTION

Significant research have, in the past, been undertaken to study the mechanical behaviour of bolted rock joints, Spang and Egger (1991), Pallet and Boulon (1993), Ferrero (1995). Bjurstrom (1974) was the first to report on the systematic research work on fully grouted rock bolts. His shear tests were conducted on fully cement grouted bolts embedded in blocks of granite. According to Bjurstrom, inclining the bolt resulted in stiffening the shearing surface by increasing the shear strength at small displacement. Dight (1982) carried out a series of laboratory tests, to evaluate the shear resistance of bolted joints using various materials and he found that the normal stress acting on the joint surface had no influence on the shear resistance. Also, joints with inclined bolts were stiffer than the perpendicular ones. Dight (1982) proposed an expression to predict the maximum forces mobilized in the bolt. He also found that the failure of the bolt was caused by the combination of axial and shear forces. Ferrero (1995) proposed a shear strength model for reinforced rock joints based on both the numerical modelling and laboratory tests. The overall strength of the reinforced joint was considered to be the combination of both the dowelling effect and the incremental axial force increase due to the bar deformation. Also, Ferrero proposed a modified analytical model for bolts installed perpendicular to the joint plane in stratified bedding plane.

The research work reported in this paper examines the interaction of rock/resin/bolt in different medium strength. The behavior of bolted rock joints was then compared with 3D FEM modeling carried out with ANSYS 3D.

2 EXPERIMENTAL STUDY

A laboratory experimental study was conducted on shearing of a bolt installed across joints in three-piece rectangular concrete blocks. Known as double shearing test, the bolts were installed perpendicular across the joints and subjected to shearing under different confining pressures. The confining pressures were affected by bolt pretension. Two different strengths of concrete blocks were cast, 40 MPa and 20 MPa, to simulate rocks of different strength. Once mixed the concrete was poured into greased wooden moulds measuring 600 mm×150 mm×150 mm which was divided into three sections. The cross-sectional area of the cast concrete is 150 mm2. Each mould had two metal dividers to separate the three sections of the concrete and a length of conduit 24 mm in diameter was set through the centre of the mould lengthways to create a hole for the bolt. The concrete was left for 24 hrs to set and then removed from the moulds and placed in a water bath for a period of 30 days to cure. The centre hole was then reamed out (diameter increase) using a drill and a hexagonal drill bit from its initial



a: testing machine

C: stress locations along the bolt

Figure 1. Photograph of tested sample in Instroning machine and sketch of deformed bolt together with a post testing deformed bolt.

diameter of 24 mm to 27 mm to simulate the real mining situations were hole diameters are typically 27 mm. A 500 kN capacity Instron servo-controlled universal testing machine, shown in Figure 1 was used for double shearing tests. The general method of testing required the application of a downward load to the centre block while the two end blocks remain stationary so as to shear the bolt as it resides within the blocks.

The concrete blocks together with the installed roof bolts were placed into a fully enclosed steel shear box and clamped down to a specially fabricated steel base platform. The base platform allowed the shear box to be mounted on to the Instron machine. Spacer plates and load cells were fitted to each end of the bolt followed by a standard roof bolt nut.

Each load cell on either side of the concrete block was connected to a read-out unit to record the tensile load generated in each end of the bolt. The bolt ends were pretensioned to a predetermined tension load, which was achieved by tightening the roof bolt nut on each end. Tensioning of the bolt ends using hollow jacks was abandoned because of the difficulty in maintaining equal loads on either side of the test block. Parameters recorded during the double shearing process include:

- 1. Shear Load supplied by Instron machine in kN;
- 2. Deflection between centre block and two end blocks in mm; and
- 3. Bolt tension/block confining loads on either side of the shear box and carried at the both ends of the bolt in kN.

Details of the experimental study are shown in Figure 1. The shear behavior of bolts in jointed concrete blocks was reported by Aziz, Pratt, and Williams (2003). Figure 1 also shows the sketch of a deformed bolt together with a post-test deformed bolt. Three bolt

	Туре		
Bolt specifications	T1	T2	T3
Bolt core dia. (mm)	21.7	21.7	21.7
Bolt dia. (mm)	24.0	24.0	24.0
Profile centres	12.00	12.50	25
UTS (kN)	330	340	340
Yield Pt load (kN)	250	256	247
Profile height (mm)	0.65	1.40	1.25
Profile angle (°)	21.5°	21.5°	21.50°
Profile top width (mm)	1.50	2.00	2.50
Profile base width	3.00	4.00	5.00

Table 1. Bolt profile details.

types were used in the study, they were known as Bolt Types T1, T2 and T3 respectively. The properties of various bolts are shown in Table 1 (see Aziz, 2004).

Tests were made with each of axial confining loads of 20, 50 and 80 KN respectively. The development of shearing loads was examined with respect to the surface bolt profile configuration. Shearing tests were conducted in different concrete strengths of 20, and 40 MPa respectively. Figures 2 and 3 show typical Shear load and displacement profiles of

two bolt types cast in both 20 MPa and 40 MPa strength concrete medium respectively. The bolts were sheared under different pretension loads.

The following points were noted from shear load and deflection graphs (Aziz, Pratt and Williams, 2003):

1. The shear load of the bolt increased with increasing bolt tension. This behaviour was obvious in bolts with low height and widely spaced profiled bolts.

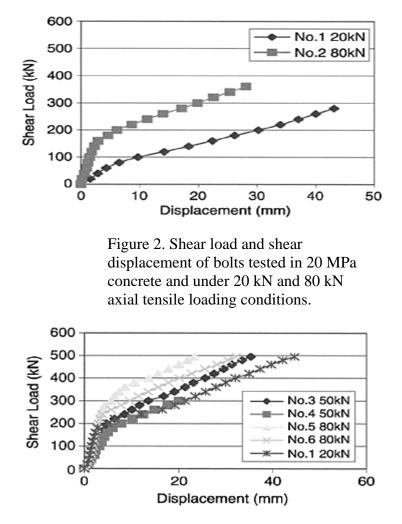


Figure 3. Shear load and shear displacement of bolts tested in 40 MPa concrete strength under 20, 50 and 80 kN axial pretensioning bolt loading conditions.

- 2. The strength of the medium has influenced the shear load level but not the trend. Shear load values for all the bolts were generally less in 20 MPa strength concrete medium in comparison to the shear load values of bolts tested in 40 MPa concrete.
- 3. The shear displacement at elastic yield point was not consistent irrespective of the concrete type and the axial load. This was the same for all the three bolt types tested.
- 4. The displacement rate at post peak yield point was relatively higher than the yield zone. This is attributed to the combination of the reduced bolt bending force as well as the reduction of the contact area of the joint surfaces. Obviously, there were some variations to this rate for different bolt types due to the bolt surface profile configurations with other parameters being almost equal.
- 5. High profiled and closely spaced bolts such as Bolt Type T2 displayed consistent shear load at all three levels of bolt tension loads in both 20 and 40 MPa concrete mediums. The consistency of shear loads at Bolt Type T2 elastic yield point was more pronounced.
- 6. Deflected bolt sections experienced regions of tension and compression. Resin columns remained adhered to the sides of the bolt region that experienced compression, but had broken off the sides that were in tension

3 3D NUMERICAL ANALYSIS

3D FEM of the reinforced structure subjected to the shear loading was used to examine the behavior of bolted rock joints in relation with the experimental results. Parameters considered were, the three governing materials (steel, grout and rock) and two interfaces (bolt grout and grout-rocks). Using ANSYS (Version 8), it was possible to simulate specifically the elastoplastic materials and contact interfaces behaviours. The process of FE analysis simulation is shown in Figure 4. The model bolt core diameter (D_b) of 21.7 mm and the grouted cylinder (D_h) of 27 mm had the same dimensions as those used in the laboratory test.

The elastic behaviour of the elements was defined by various material Young's Moduli and Poisson's ratio, as per Table 2. The stress-strain relationship of the steel was assumed as bilinear kinematics hardening model and the modulus of elasticity of the steel was accounted as a hundredth of the original value (Cha and Choi, 2003).

The yield strength of the steel of 600 MPa was obtained from the laboratory tests. The results of both the numerical modeling and the experimental results, in different rock strengths and different bolt pretensions, are shown in Figures 5 to 11. It can be seen that the numerical simulations were found to be in close agreement with the experimental results.

Figure 11 shows the relationship between shear displacement and rock strength obtained from both the laboratory tests and numerical simulation analysis. Clearly, good correlations are evident for both the numerical simulation and the laboratory studies. Figure 12 shows the comparison of pretension effect on bolt ends. Figure 13 shows the behavior of bolt during loading.

The interface behavior of grout-concrete was considered as a perfect contact, and was determined from the test results. However, a low value of cohesion (150 kPa) was adopted for grout-steel contact. The 3D solid elements (Solid 65 and Solid 95) that have 8

nodes and 20 nodes were used for each of concrete, grout and steel elements respectively. Each node had three translation degrees of freedom, which tolerated irregular shapes without significant loss in accuracy. 3D surface-to-surface contact element (contact 174) was used to represent the contact between 3D target surfaces (steel-grout and rock-grout).

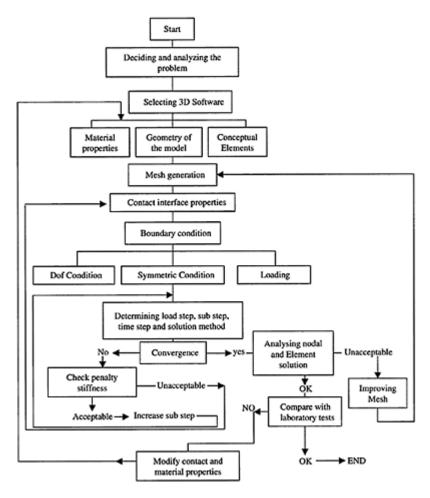


Figure 4. The process of FE simulation.

Table 2. Elastic material properties.

Material	UCS (MPa)	E (GPa)	Poisson's ratio
Concrete	20	20	0.2
Concrete	40	32	0.2

Resin grout	60	12 0.25
Steel	-	200 0.3

This element is applicable to 3D structural contact analysis and is located on the surfaces of 3D solid elements with mid-side nodes. The numerical modelling was carried out at several sub-steps and the middle block of the model was gradually loaded in the direction of shear. Simulation of several models in varying conditions (a range of bolt tensile load and

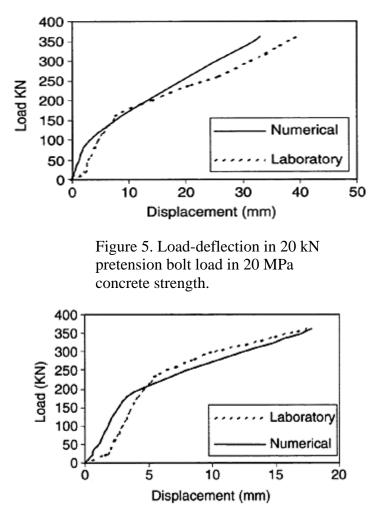
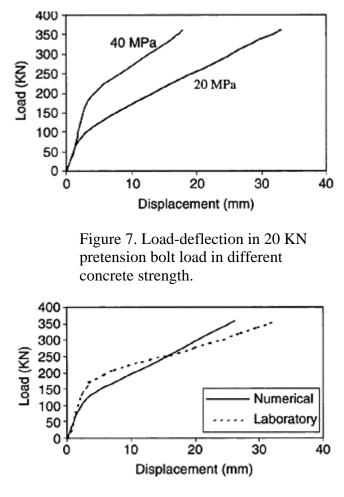
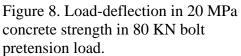


Figure 6. Load-deflection in 20 KN pretension bolt load in 40 MPa concrete strength.





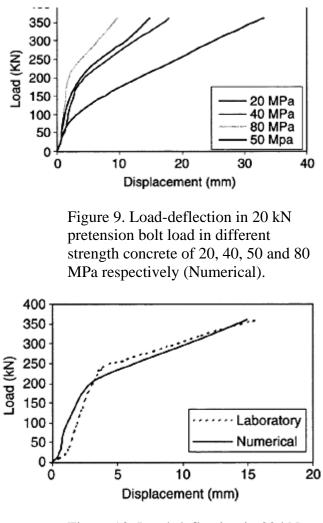


Figure 10. Load-deflection in 80 kN bolt pretension load in 40 MPa strength concrete.

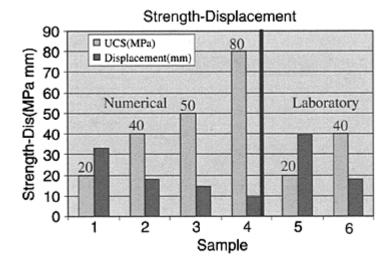


Figure 11. Comparison of numerical modelling with experimental tests on the effect of rock strength on shear displacement.

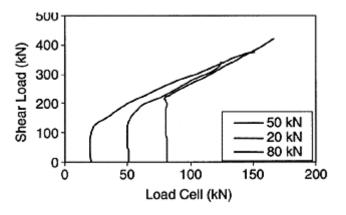
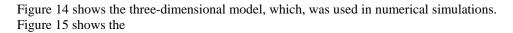


Figure 12. Shear load and load cell readings for various bolts with initial loading of 20, 50 and 80 kN respectively.

concrete strengths) was carried out under a vertical load and the results were analysed for both linear and nonlinear regions of the load-deflection curve.

4 RESULTS AND DISCUSSION



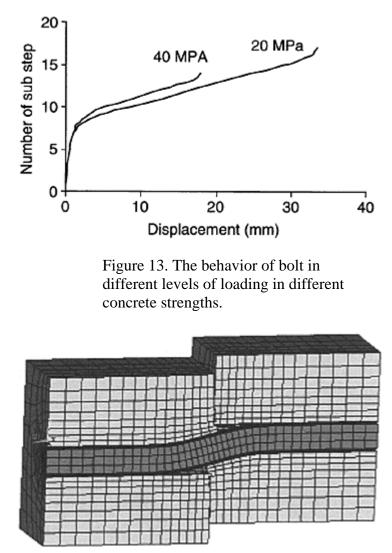


Figure 14. Three-dimensional and deform shape of the numerical model.

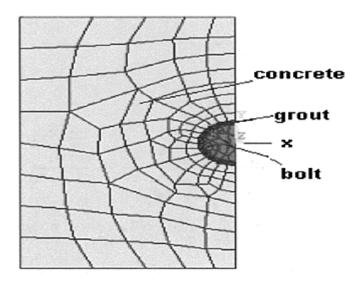
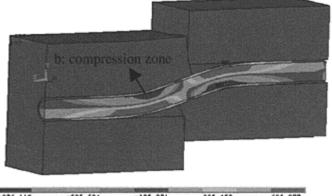


Figure 15. Cross-section of the numerical model.

model cross section. Figures 16 and 17 show the stress contours along the bolt in 20 MPa strength concrete at axial pretension loads of 20 kN and 80 kN respectively.

The strength of the concrete has shown significant changes in shear displacement. Shear displacement



-976.116 -605.694 -126.071 226.469 605.977 -620.056 -320.332 50.191 420.715 791.23

Figure 16. The stress contours along the bolt in 20 MPa strength concrete with 20 kN in post-failure region.

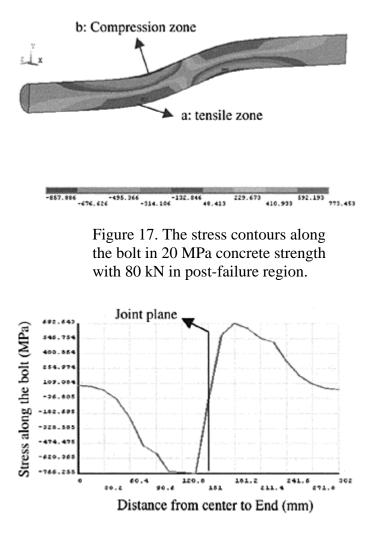


Figure 18. Stress changes along the bolt in upper fiber.

has decreased with increasing the strength of the surrounding rock. Increasing the confining pressure through bolt pretension, caused a reduction in bolt deflection, however this reduction, when occurred prior to the elastic yield point, was shown to be insignificant, as demonstrated in both the experimental and numerical results in Figures 5 to 11.

The effect of bolt pretension in the yield zone has significantly affected bolt deflection this was demonstrated in both the numerical and the experimental results. As can be seen in Figure 16 and 17 the stresses at the upper convex side of the bolt at "a"

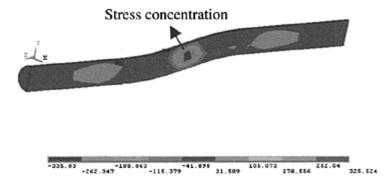


Figure 19. Shear stress contours along the bolt with 40 MPa concrete strength in post failure region and 20 kN pretension bolt load.

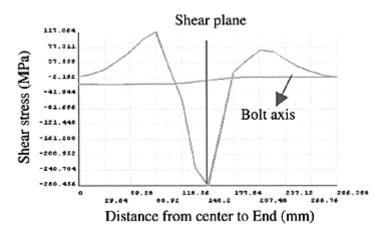


Figure 20. The rate of shear stress changes along the bolt with 40 MPa strength concrete and 20 kN pretension bolt load.

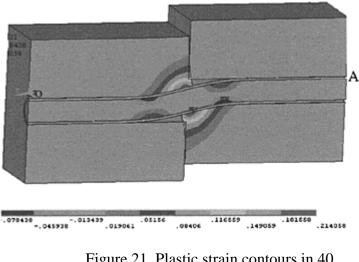


Figure 21. Plastic strain contours in 40 MPa strength concrete with 20 kN pretension load.

and towards the perimeter were tensile while it was compressive at the concave side "b". However the stress conditions at the lower opposite section of the bolt was reverse. The value of these stresses is higher than steel yield point.

The degree of the stress changes in post failure region is plotted in Figure 18. This phenomenon was also observed from the experimental test results shown in Figure 1. It can also be seen that the stress levels in these zones were high and the bolt appears to be in

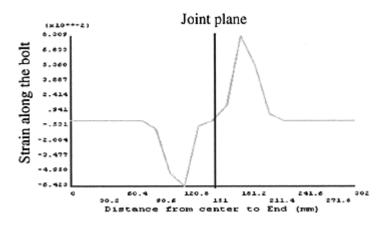
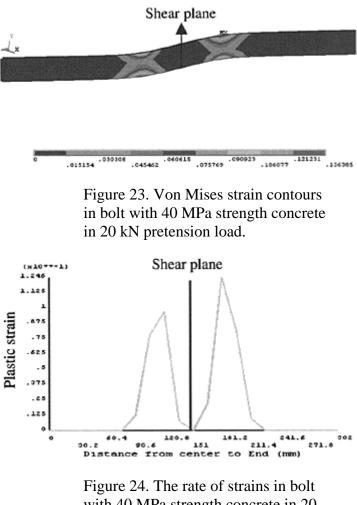


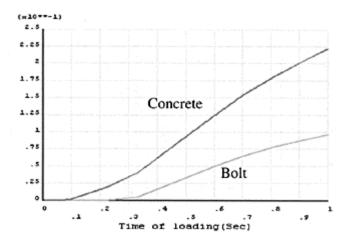
Figure 22. The rate of strain changes along the bolt in 40 MPa strength concrete with 20 kN pretension load.

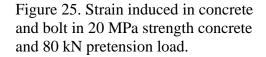


with 40 MPa strength concrete in 20 kN pretension load.

yield state. Figures 19 and 20 show the shear stress contours and stress rate along the length of a 20 kN load pretensioned bolt installed in 40 MPa strength concrete medium. The maximum shear stress is concentrated in the vicinity of the joint plane.

Figures 21, 22, 23 and 24 show plastic and von mises strains and the rate of strain change along the bolt in 40 MPa strength concrete with 20 kN pretension load. As Figure 25 shows, the bending of the bolt becomes predominant at small rate of loading and concrete has started to produce strain around 10 percent of loading period and these strains were remained permanent with increasing the shear force. Steel bolt has





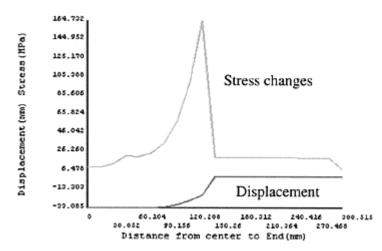


Figure 26. Stress changes in concrete with increasing deflection in 20 MPa strength concrete and 20 kN.

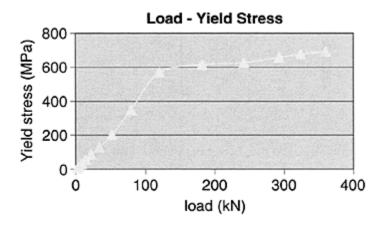


Figure 27. The rate of yield limit of the bolt versus shear force in concrete 20 MPa with 20 kN pretension load.

begun to create strain around 25 percent of loading period and it has appeared in yield situation after 40 percent of loading period. Figure 26 shows the stress changes in 20 MPa strength concrete with increasing shear load in 20 kN pretension load.

Contact pressure contours were found to increase with increasing the shear load and visa versa. However, the increase of pretension has reduced contact

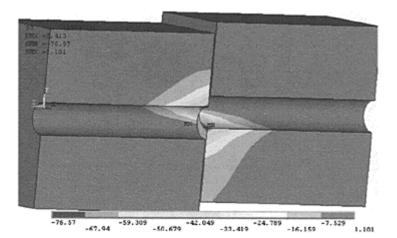


Figure 28. Min stress contours in concrete 20 MPa strength and 20 kN pretension load in pre-failure region.

pressure. As the Figure 27 shows the yield limit of the steel is reached very quickly at about 0.33 P and 0.45 P in concrete 20 and 40 MPa respectively. Further increase of the shear force almost has no apparent influence on the stresses in the hinges points.

Figure 28 shows induced stresses in concrete blocks corners, which are affected by the steel pressure, when bending.

As it can be recognized, the stresses around the edges in this area are high and higher pressure can induce longitudinal fractures in concrete blocks. This was also observed by the experimental results.

5 CONCLUSION

The numerical simulation of shearing the reinforced joints and bedding planes provides a unique insight into the build up and distribution of stress when shearing occurred. A number of conclusions drawn based on the numerical solutions were found to be in agreement with the experimental results. Harder medium of concrete has shown significant effect on reduction of shear deflection. As strength of material increased four times it shows approximately four times reduction in the shear displacement, it means the relationship between rock strength and shear displacement is directly inverse. The maximum deflection was observed in softer medium and lesser initial tensile load. The higher value of tensile and pressure stresses are located on each side of the shear plane and increase with increasing shear load.

However, after yield point there is no significant changes in them, that these locations were observed in experimental test. A higher value of shear stresses was very concentrated near the shear plane in bolt. Increasing pretension load causes reduction in shear load. However, after yield point shear stress for all situations approximately is constant. Concrete in critical contact areas because of much pressure starts to break and this pressure produces tensile stresses through the concrete and then concrete finds longitudinal fractures. This situation was observed in both experimental and numerical results.

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Research on new anchoring method by physical modelling and field testing

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ABSTRACT: A comprehensive comparison on reinforcement effect of rock bolts with different parameters and applied forms by physical modelling for rock block has been conducted. Then a comparison on effect of different applied method of rock bolts for tunnels with two types of cross section by physical modelling was made. Finally a field application of the new reinforcing method was conducted in a testing tunnel of a deep coal mining. Based on the testing results a new reasonable scheme of applied method for rock bolts called "inclined and crossed applied method" was suggested to engineer.

1 INSTRUCTIONS

To excavate a tunnel in soft rock of mining with great depth is usually a difficult task. Therefore, how to design a new scheme to reinforce the surrounding rock of a tunnel effectively, which is excavated in weak rock mass is a very important research topic. So we will discuss these problems.

2 SIMULATION TESTS OF REASONABLE ANCHORING METHOD OF SURROUNDING ROCKS

In the case of fairly high initial geostress and weak surrounding rocks, for example, in China, the convergences of walls or between the roof and floor of an underground opening in some mines is even as high as 40–100 cm. Undoubtedly the key problem lies in introducing a reasonable anchoring scheme to increase the strength of rock mass and

decrease too large deformations, this is true for either temporary support or permanent support. Nowadays, the anchoring angles in traditional methods are all perpendicular to opening surfaces. The author has carried out a systematic research on the effects of various anchoring angles and densities and obtained some significant conclusions.

2.1 Preparing the new file with the correct template

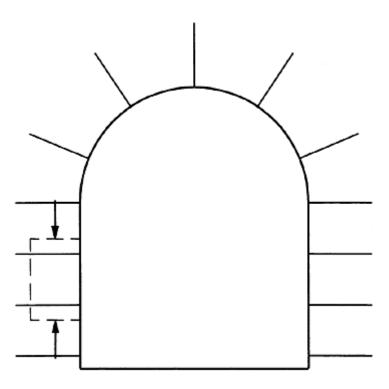
To simulate the soft rock and effect of bolting more truly, the most important task is at first to compound an equivalent modelling material possessing a good analogy property. This work has been published elsewhere [Zhu, 1989 and 1995].

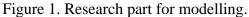
A separate rock block was taken from the sidewall of an opening (Figure 1) and a great number of bilateral loading tests were performed on the reinforced block to study the effects of different anchoring parameters. The dimension of the testing block is $10 \times 10 \times 20$ cm³. The loadings to the testing block are σ_1 , σ_2 , and σ_3 respectively, which simulate the stress state at the opening surface as tangential stress, confining one along the tunnel axis and radial one approximately chosen as zero. ε_2 (=0) was remained constant in testing, while σ_1 and σ_2 were increased until the tested block failures and the complete stress curves including the post failure curves were obtained.

The effects of various simulating bolt materials, anchoring types, anchoring densities and anchoring angles were compared with each other. A kind of bamboo ($E=1.0\times10^4$ MPa, diameter=2 mm) and plexiglass ($E=0.3\times10^4$ MPa, section sizes= 2×2 mm) were used as bolts.

Two anchoring types were employed, i.e. fully column and end-bonded types; the anchoring densities were those of 8, 10, 12, 36 rods per 200 cm² respectively (Figure 2), which were corresponding to 0.4 m–1.2 m of the real bolting intervals in the walls of tunnel. The comparison of the effects between different anchoring types-bolt applied normally to sidewalls, bolt inclinedly crossed in perpendicular planes, bolt inclinedly crossed in horizontal planes was made (Figure 2). In the same figure, the comparison of various anchoring angles can be seen as well.

Listed in Table 1 are the testing results of various anchoring types with different parameters. The loadingstrain curves are indicated in Figures 3 and 4.





2.2 Interpretation and analysis of testing results

Under uniaxial compression, the peak strengths of testing block with bolts are, compared with no bolts, increased by about 17% and the residual strength is increased by about 100%; the tensile strength for Brazilian tests is increased by one time or so. As for biaxial compression, the peak strength is increased by

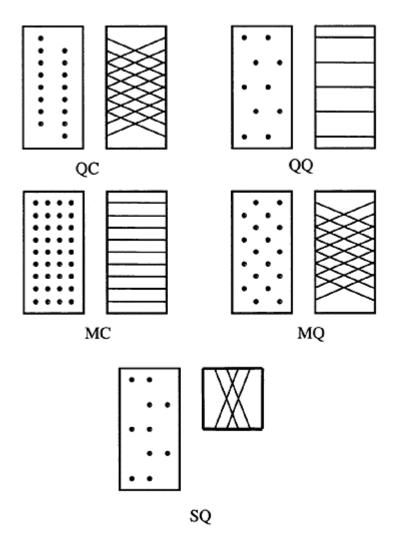


Figure 2. Different schemes of bolt installation.

		testing.					
Туре	Symbol	Density/ direction	Bond type	Soften character	Bolt material	Peak strength (MPa)	Ratio σp/σS
C1	Uniaxial comp.	0		Yes		1.0	1.0
C1	Biaxial comp.	0		Yes		1.3	1.3
C2	SQ-2	12/75	Full column	Yes	Plesi-glass	1.5	1.5
C3	MQ-2	12/68	Ditto	No	Ditto	1.4	1.4
C4	DC	10/90	End fixed	Yes	Bamboo	1.5	1.5
C5	QC	10/90	Full column	No	Ditto	1.5	1.5
C6	QQ	8/73	Ditto	Yes	Ditto	1.54	1.54
C7	SQ	12/75	Ditto	No	Ditto	2.06	2.06
C6	MQD	12/68	End fixed	No	Ditto	2.1	2.1
C6	MQ	12/68	Full column	No	Ditto	2.2	2.2
C5	MC	36/90	Ditto	No	Ditto	3.0	3.0

Table 1. Comparison of different bolt type for testing.

Note: The density means the number of bolts per $0.02m^2$ and the direction indicates the angle between the bolt and the wall.

50% to 100% and even more (Figure 3). The anchoring density effect is also shown in Figure 3, the peak strength of curve MC (high density) is even increased by 3 times (with no dilatancy).

The strength of specimens relates to anchoring density as well to anchoring angles, anchoring types, shearing resistance and rigidity of simulating bolt materials.

The anchoring type of inclined and crossed ones is remarkably favourable for increasing peak strength and controlling dilatancy of rock masses. The excellent anchoring effect is obtained when bolts meet at about 68 with the free surface of an opening. Since this kind of bolt arrangement needs strong resistance capacity, so the bolts are required to have fairly high shearing resistance and lateral rigidity.

Anchoring types (fully-column or end-bonded) have no obvious effect on the peak strength of rock masses. However, the volumetric strain curves of these two types are different from each other. In the case of fully-column bolts, the volumetric curve is more plentifully and the dilatancy appears much later,

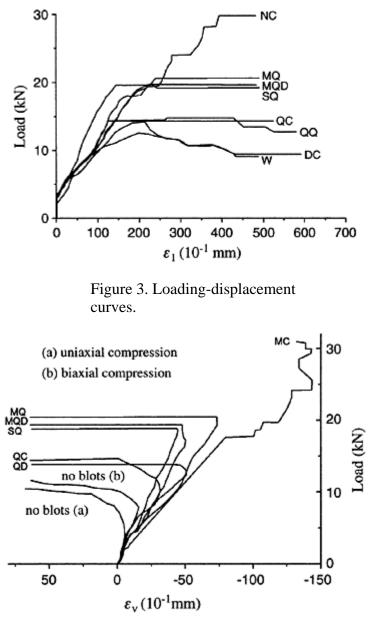


Figure 4. Loading-volumetric deformation curves.

its action of governing dilatancy is rather effective. In addition, the simulating rock block with fully-column bolts basically presents no softening phenomenon. In contrast, this

phenomenon takes place in the case of end-bonded bolts, so the post failure strength is lower.

3 ENGINEERING MODEL SIMULATIONS FOR ANCHORAGE EFFECTS OF GALLERIES

In order to verify the effect of tests on the anchored blocks above mentioned, this present section will describe the model simulation tests of anchored galleries. The reader is referred to reference (Zhu, 1989 and 1995) for the composition and mechanics character of similarity model's materials.

By consulting the conditions of a deep gallery being under excavation in sedimentary rock strata, the support of galleries under consideration can be regarded as the problem of both stability and support of galleries in soft rocks subject to a fairly high stress field, because the surrounding rocks are in the affecting zone of faults. The initial stress field is approximately taken on gallery's horizontal section as $\sigma_x=19.5$ MPa, $\sigma_y=12.8$ MPa and rocks have a uniaxial compressive strength of $\sigma_c=30$ MPa, a Young's modulus of E=8-10 GPa and a Poisson's ratio of $\mu=0.25$ respectively. The ratios of stress and geometry for similarity are assumed as 40 and the maximum sizes of the whole model are 50×50 mm.

In the light of match principle in rigidity, the bamboo sticks measuring $\oint 2 \times 40 \text{ mm}$ in sizes are used. The mortar for full-column cementation of bolt is simulated using white latex. Two patterns of circle and horse hoof-shaped opening cross-section (Figure 5) are adopted in model simulation. The modelling tests are conducted in the plane stress state. In testing, three kinds of bolt arrangement schemes are designed: (a) inclined-crossed installation of bolts (Figure 6), installing direction making an angle of $\alpha=\pm 22.5^{\circ}$ with the normal direction of gallery periphery (Figure 6); (b) installing of bolts along the normal of periphery (Figure 6) and (c) no bolts. The bolts are embedded in the predicting location during preparing the model.

The testing process is as following: firstly, keep the vertical stress two times as high as the horizontal stress and then increase stepwise loads along the two directions simultaneously with an increment of 0.1 MPa until the horizontal and vertical stress components reach the levels of 1.0 MPa and 2.0 MPa respectively. In the case of gallery with horse hoof-shape scheme, increase the vertical stress continuously until the 4.0 MPa being reached with a constant horizontal stress remained. In the loading process, record the data of convergence between the roof and floor and two sidewalls. The testing results have shown (Figures 7 and 8) that:

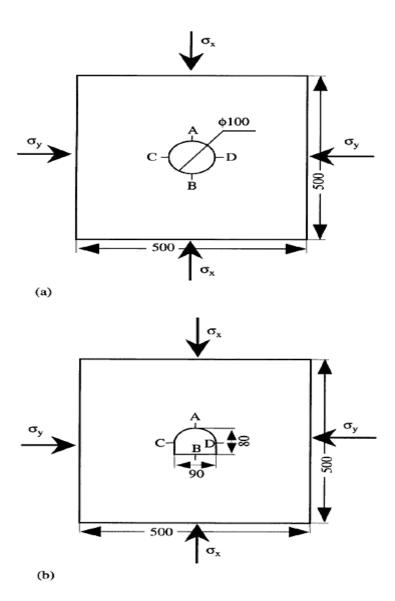


Figure 5. Structure and size of engineering model. (a) circle gallery; (b) gallery with horse hoof-shape. AB is the alignment for convergence measurements of roof and floor plates; CD is the convergence alignment of two sidewalls. (1) For either gallery with circle shape or horse hoof-shape, the convergence deformation of surrounding rocks reinforced using inclined and crossed arranging scheme of bolts is markedly lower than that reinforced using normal arranging scheme. For instances, the convergence between roof and floor of former one is reduced by 20–40% and those of the two side walls are reduced even by

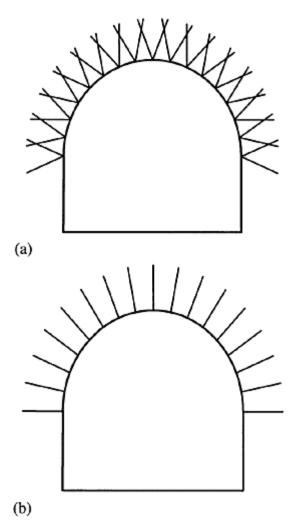


Figure 6. Two schemes of bolt installation. (a) normal installation and (b) inclined and crossed installation. over 60%; and the convergence between the roof and floor of latter's gallery is reduced by 16–40% and the plastic rupture wedge formed in two side walls is small.

(2) The comparisons between normal bolting support and no-bolting support, the former has a smaller convergence obviously than the latter, being able to control the development of the plastic wedge. In the case of circle shape gallery, the convergence between roof and floor is reduced by 33–45% and that of the side walls by 50–80% and in the case of gallery with horse hoof-shape, this reduction in the convergences between roof and floor is about 14–43%.

4 IN SITU TESTING

In order to verify the results of model simulations in lab, in-situ tests have been conducted in the gallery of a coal mine. At first, a number of types of extension bolts and flexible shotcrete material were developed, being applicable to large deformations of rock surrounding, then field tests were performed in the testing section of a gallery and other useful observation work was also made. This is described below briefly.

4.1 Development of new bolts applicable to large deformations of rock surrounding

Since the bolts are installed in the weak surrounding rocks where abrupt deformation may take place, the new type of bolts should have the following performance:

- (1) have an initial and a final reinforcing force that are high enough;
- (2) be applicable to large rock deformations but not ruptured and have ability against separation from rocks;
- (3) ensure a high lateral bending resistance;
- (4) have a long service life and low cost.

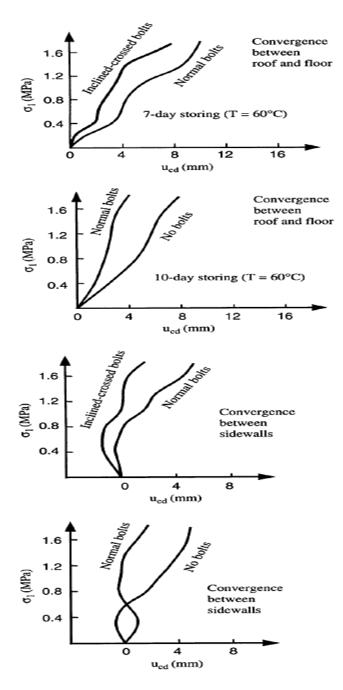
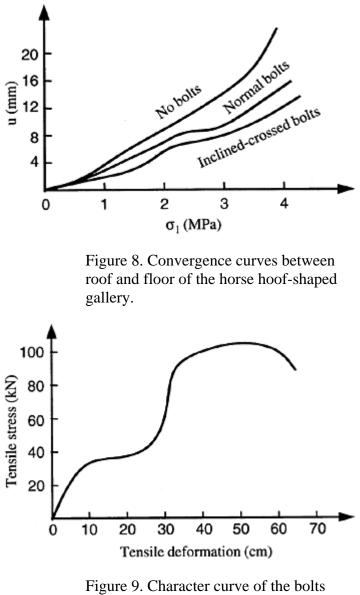
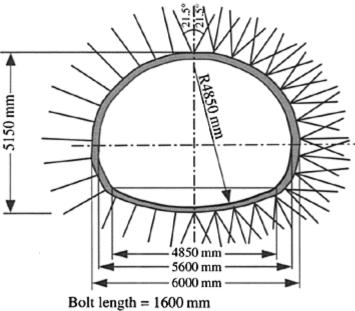


Figure 7. Convergence curves of the circular-shaped gallery.



being full column bond type.

The authors of this paper have successfully developed several kinds of bolt that roughly meet the above mentioned requirements. For example, one is column compound structure type and the other is collar-extension structure type. Figure 9 gives the characteristic curve of the bolt of compound structure type. It can be known from the figure that the initial anchorage force is over 3 ton and the limit load is over 60–100 kN. In early stage, the limit deformation



Bolt length = 1600 mm Primary soft shotcrete thickness = 50 mm Secondary soft shotcrete thickness = 50 mm Normal soft shotcrete thickness = 100 mm

Figure 10. Layout of different reinforcement methods for gallery section.

can be high up to 20 cm and it is even higher in the second stage.

4.2 Field testing of the new type of reinforcement

The extensionable bolts described in the above section have been used in a main transportation tunnel of a coal mine with 640 m depth, located in the affecting zone of faults, to test the supporting effect. Shown in Figure 10 are the gallery cross section and two supporting design schemes. The rocks surrounding of the gallery belongs under the affection zone of faults with joints developed, the global strength being low and the horizontal tectonic stress approximating the vertical stress. The rocks surrounding of the testing section are the interbedded strata of sandstone and shale.

The on-time full-face-preruptured-smooth blasting is made in excavation and the joint support of bolting-shotcrete-network is adopted. The inclined-crossed arrangement scheme of bolts is used and the flexible shotcrete layer is made. The network is made of steel bars with net size of 150×150 mm. The secondary support (100 mm thick common concrete shotcrete) is conducted after the gallery becomes stable.

Some useful field monitoring work has been also carried out in the advance and supporting process, such as (a) convergence measurements, (b) extensioneter measurements, (c) measurements of bolt's deformation and (d) acoustic wave measurements of relaxation zones of the rock surrounding.

Here, only the measuring curves of displacements of surrounding rocks by extensioneters are given (Figure 11). It can be seen from the Figure 11 that the

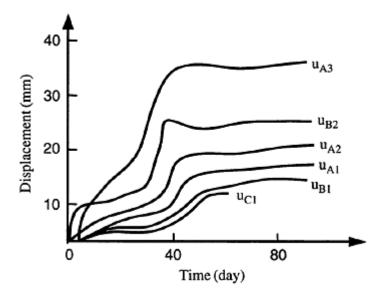


Figure 11. Displacement curves measured by extensioneter at the gallery.

rock deformation tends to stability roughly in 40 days. Accordingly, the secondary shotcrete work can be made one month or so later after the primary support finishes.

Although aforesaid gallery has experienced operation for more than 10 years, its stability status is good. Whereas large deformation and rupture phenomena repeatedly take place in the adjacent galleries, which shows that the new reinforcement method proposed in this present paper is effective.

5 CONCLUSIONS

The model simulation test is a good method that is feasible to study the reinforcement effects of various anchorage ways for galleries in soft rocks. The model simulation tests on an engineering gallery built using model blocks have shown that the strength of surrounding rocks can be increased greatly but their deformation is reduced providing the new inclined-crossed applied method of bolts is used.

Field tests have further verified that the new method of reinforcement is effective and useful and it can make the rock deformation become stable in a long time.

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5 Open pit

Highly flexible catch fences and high performance drape mesh systems for rockfall protection in open pit operations

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ABSTRACT: Rockfall hazards in open pit applications mainly occur in steep open pit walls due to aggressive pit design, in flat walls without berms while following shallow dipping ore bodies or locally on batter level. Areas with high damage potential such as decline portals or haulage ramps are especially hazardous. Dangers from falling rocks have to be reduced as much as possible. The protection systems to cope with such hazards from the manufacturer Geobrugg which are described in this paper are highly flexible and consist of high-tensile steel components. They are field tested and are thus rated with a certain energy absorption capacity. In order to get impact velocities and energies, a rockfall simulation is run, utilising actual slope characteristics. This study deals with three case studies of rockfall protection systems recently implemented in Western Australia. The first study describes a portal protection fence, the second one a ramp protection fence and the third one a high-performance drape system with impact section.

1 INTRODUCTION

From a global point of view rockfalls in open pit mines mainly occur in steep open pit walls due to aggressive pit design or in bermless flat walls while following shallow dipping ore bodies. Locally, rockfalls can take place on a batter level, with rocks falling out of the crest or the batter itself and reaching the next berm. In areas with a high danger potential such as portals, ramps, escape ways or exhausts, adequate measures have to be taken to reduce this risk to the work force and infrastructure. To cope with such hazard, different strategies and solutions can be implemented such as increasing berm width, building windrows, installing drape mesh or rockfall barriers. Rockfall barriers have the advantage that if a stable foundation can be achieved, it is possible to effectively protect the area below the fence without the need to access onto the berm and without enlarging berms. This is mainly an interesting solution due to the fact that the overall angle is not affected or can be developed even steeper.

In order to be able to absorb energies in the range of 100 to several thousand kJ, it is necessary to install flexible systems which gradually slow the rocks. Systems made of steel beams, sleepers or wire ropes are too stiff and thus induce large forces on the barrier during the impact. Due to the non-linear behaviour of flexible rockfall barriers it is still not possible to design them on paper only; they still have to be tested "one on one".

The Swiss company Geobrugg is a leading supplier of rockfall protection systems. Geobrugg gained experience in this field over decades and has implemented hundreds of projects worldwide. The systems are all field tested to determine the energy absorption capacity. The projects described here were installed by Rock Engineering, a ground support specialist company based in Perth.

2 HIGH-TENSILE WIRE MESH

The main element of the protection systems presented in this paper is the high-tensile chain-link mesh called TECCO. The mesh was developed by Geobrugg not only for static applications such as slope stabilization but for dynamic applications such as rockfall protection catch fences or for use as drape mesh as well. Both the static and dynamic performance of the mesh is proven by independent testing and applications.

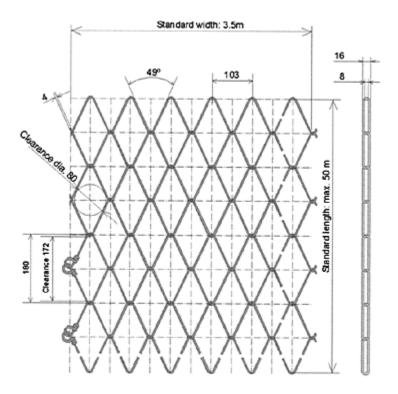


Figure 1. Geometry of the TECCO mesh.

Table 1. Properties of the TECCO mesh G-80 4 mm.

Clearance diameter	80 mm
Wire diameter	4 mm
Wire strength	1770 MPa
Breaking load of a single wire	22 kN
Tensile strength longitudinal	200 kN/m
Tensile strength transversal	80 kN/m
Weight	2.6 kg/m2

The mesh is made of high-tensile steel wire with a diameter of 4 mm and a tensile strength of 1770 MPa. Furthermore, this high-tensile wire has an excellent shear resistance. The mesh is diamond shaped and the wires at the selvedge are bent over and double twisted in such a way that this connection is as strong as the mesh itself. The mesh

is produced in rolls and can be manufactured in widths up to 4 m and in tailor-made lengths.

The static strength of the mesh was determined in several laboratory testing programs by Torres (2002) at the University of Cantabria in Santander, Spain. The characteristics of the mesh are summarized in the Table 1.

The tests of Torres (2002) further showed that there is no unravelling of the mesh once a wire failed. The tests were executed with a wire cut before testing the longitudinal strength. The test panel reached the same breaking loads with or without a cut wire.

3 DYNAMIC TESTING OF THE MESH

Several dynamic testing programs were carried out with the TECCO mesh on the test site of the Swiss

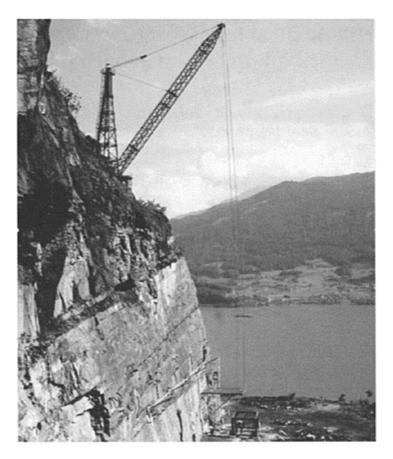


Figure 2. WSL test facility Walenstadt, Switzerland.

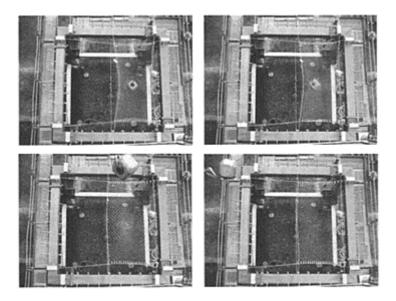


Figure 3. 50 kJ test with a 2 m×4 m panel of TECCO mesh.

Federal Institute of Forest, Snow and Landscape (WSL) in Walenstadt, Switzerland. The tests were executed by Geobrugg under supervision of WSL.

The testing facility consists of a steel frame construction with a crane installed above, with a remotely controlled release hook. In the test frame mesh panels of up to 4 m×4 m can be put in place at a height of 5 m above ground. The tests are executed by using concrete boulders of different sizes released from a predetermined height (up to 65 m). Load cells are put

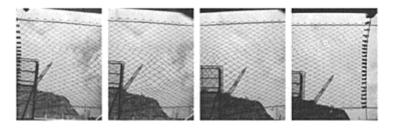


Figure 4. TECCO mesh after 50 kJ test.



Figure 5. Impact testing on drape mesh.

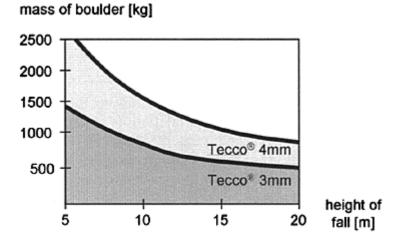


Figure 6. Results obtained from drape testing.

in place under the frame and in the support ropes and two high-speed digital video cameras (250 frames/sec) record the impact sequence.

Sennhauser (2002) executed several 50 kJ tests in late 2002 with a 2 m×4 m panel of TECCO mesh. The impact energy was achieved by releasing smaller blocks from greater heights and also larger blocks from lower heights. The mesh stopped all the blocks and consequently showed the performance to easily absorb 50 kJ. The braking distance was in the order 1.5 m.

The mesh did not show any damage. It was slightly deformed but there were no broken wires (see figure 4). Trials with multiple impacts in the same location of the mesh proved that the mesh is able to absorb at least two impacts of 50 kJ. Due to the very short braking distance, the forces in the support infrastructure get naturally quite high. But since these forces are known, it is possible to dimension ropes, posts and anchorage.

In addition to rockfall testing, Geobrugg also carried out field tests with high performance drape mesh. In these trials TECCO mesh was used with wire diameters of 3 mm and 4 mm. In order to simulate the impact of a block into the mesh while sliding

down behind the curtain, the mesh curtain was installed with an angle of 30° between the mesh and rock face and then the blocks were released into it (mirrored situation). Figure 5 shows such a test with 1730 kg free falling from 5 m into the drape mesh.

By using this setup it was possible to determine the capacity of the mesh and this makes it possible to dimension a drape mesh system. The results are summarized in figure 6.

Compared to other mesh products, the TECCO mesh has the advantage that its properties and performance are very well established (statically and dynamically).

4 ROCKFALL SIMULATION

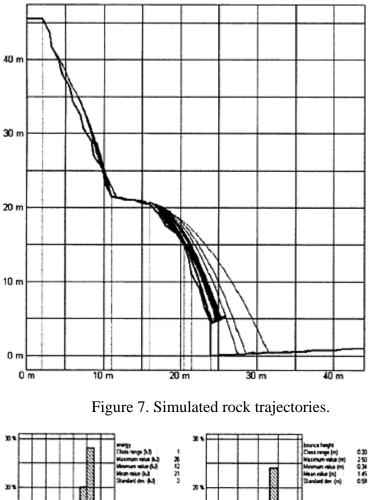
In order to determine energy requirements in a specific case, special software is needed. The potential block sizes can be estimated by site visits but the impact velocities and kinetic energies have to be simulated. In the simulation described here the software is called ROCKFALL and was developed by Spang (1995) in Germany.

Firstly, the cross section of the simulation has to be defined and split in slices with different properties such as dynamic and static friction, damping factors, rolling resistance and surface roughness. All these values are defined probabilistically with standard deviations. The blocks can be chosen as spheres or, more realistically, as cylinders.

The simulations calculate the trajectories of the blocks by using the laws of kinematics and linear and triangular momentum. The movement of the rocks can be bouncing, rolling, sliding or toppling. The iterations are repeated until the block strikes a protection structure or comes to a complete stop. Figure 7 shows the cross section and the calculated trajectories of case study 1.

Since the characteristics of the ground conditions follow certain distributions, there is for every block a different trajectory and there is a distribution for impact energies and bouncing heights as well. The software is able to plot these distributions and by considering them, it is possible to choose a rockfall barrier with a certain capacity within a chosen level of confidence. Figure 8 shows the energies and bouncing heights according to the trajectories of figure 7.

By field testing the rockfall barriers, the energy absorption capacities can be determined, and by executing rockfall simulations of a specific site, its demand of energy absorption capacity can be evaluated.



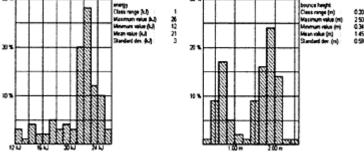


Figure 8. Obtained distribution of energies and bouncing heights.

5 CASE STUDY 1: PORTAL PROTECTION

Portals are key elements for underground operations and thus the risk that a portal could be blocked has to be kept as low as possible. For that reason most portals are highly reinforced and supported with bolts, mesh and shotcrete. If there are no stability problems in the wall but the danger of rocks falling out of the face or crest above, a rockfall catch fence above the portal can be an effective and cost-efficient alternative to meshing or shotcreting the wall.

A first application of this kind was realised at the 10140 portal in the Kanowna Belle Gold Mine near Kalgoorlie in Western Australia. Since this portal is not the main portal to the operation and is used at the moment just to access a fan and pumps, it was an ideal location for a first test installation. The requirements for the catch fence were to stop falling rocks, to be installed easily and quickly and to be maintenance free.

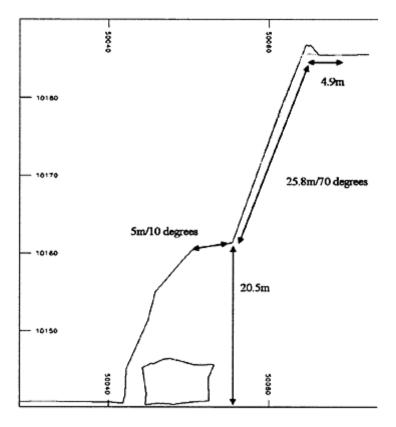


Figure 9. Cross section of 10140 portal.

The maximal block size was identified to be about 30 to 50 kg falling out about 40 m above the portal. The according rockfall simulation (see figure 7) showed that a

maximum velocity of 20 m/s has to be expected which results in a maximal kinetic energy of 10 to 15 kJ. A system height of 3 m was regarded as being sufficient.

To meet the requirement of a maintenance free fence, a chevron type design was chosen. The basic idea is that the falling rocks hit the catch fence and are rejected to one or the other side of the portal.

The fence consists of 5 posts made of thick steel tubes which were grouted into the rock by using large but short airleg holes (65 mm diameter). In order to reduce bending moments in the posts, the post heads are anchored back to the face by retaining ropes. These ropes are connected to eyebolts which were grouted into the rock. To hold the mesh in place and carry the loads to the posts, wire ropes were attached to the posts as can be seen in figure 10. Finally the TECCO mesh is assembled to the support ropes by using shackles.

The installation of the fence took place in June 2003 and took five days for a two men crew. Firstly the holes were drilled and the posts and eyebolts grouted in. Afterwards the retaining ropes were attached to the eyebolts by using wire rope clamps and then the support ropes were added to the posts. The mesh was prepared in such a manner that it was possible to put it on the centre post and be released on both sides. After the fixation of the mesh by shackles, the retaining ropes were attached to the post heads and then tensioned.

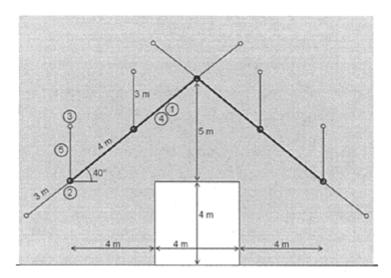


Figure 10. System drawing of the portal fence.

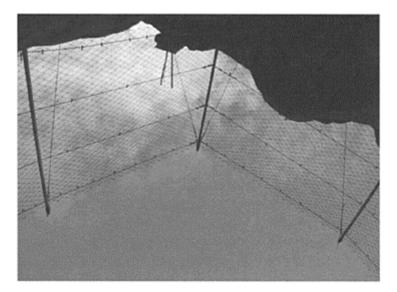


Figure 11. Installed portal fence at Kanowna Belle.

This type of portal protection proved to be an effective and cost-efficient protection system and can be a real alternative to extensive meshing or shotcreting. This makes is not only feasible for permanent portals but especially for temporary ones. Besides portals, further applications can be the protection of escape ways, exhausts or adits.

The energy absorption capacity of this design is in the range of 20 kJ. For higher energies the high-tensile mesh has to be combined with stronger infrastructure and then energies up to 50 to 100 kJ are achievable. For even higher energies systems with ring nets are available with absorption capacities of up to 3000 kJ.

6 CASE STUDY 2: CATCH FENCE

As mentioned above, engineered catch fences are a feasible and designable measure to protect people and infrastructure from the hazard of falling rocks. This case study deals with a catch fence which was installed along a ramp in the Fimiston Superpit of Kalgoorlie Consolidated Gold Mines in Kalgoorlie, Western Australia.

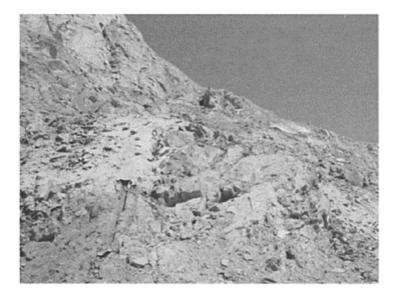


Figure 12. Area to be addressed.

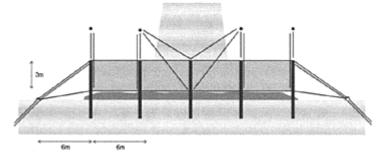


Figure 13. System drawing of catch fence.

The area to be addressed here is an old, filled stope of about 20 m in width, which was exposed. Rocks at the crest above it threatened to fall out and hit the haul ramp, endangering heavy transport and light vehicles alike. In order to deal with this hazard, a 24 m long and 4 m high catch fence was considered. The energy absorption capacity was determined to be in the order of about 50 kJ.

The implemented system consists of 5 universal columns with a post spacing of 6 m. These posts are 3 m long and are connected to heavy duty steel tubes which were introduced in 165 mm diameter holes. The tube to column join sat one meter above finished ground level. In this bottom part of the fence a 1 m high windrow was considered to get the 4 m system height together with the 3 m high mesh. In order to be

able to anchor the cables in solid rock, all the anchors are placed on both sides of the filled stope and not into it.

The posts are anchored back to the slope by using wire ropes and twin-strand cable anchors with a loop at the surface. These anchors have the advantage of a



Figure 14. Installed catch fence along the ramp.

flexible head and are thus insensitive to shear. The mesh is held in place by a top and a bottom support rope as well as vertical ropes along the posts.

The installation of this catch fence was done in December 2003 and took 3 days for a two man crew. After drilling the holes, the tubes and cable anchors were put into place and grouted in. Universal columns were then bolted to the tubes and tie back and support and ropes attached and tensioned back. Mesh was then fixed to support ropes by weaving the top and bottom ropes into the chain link of the mesh. Mesh was then connected to the vertical ropes with rope clamps. The windrow was filled in just prior to the area being reopened to passing traffic.

This example shows that it is easily possible to install a quite strong catch fence without need of access to a batter. The catch fence will protect the ramp in this area from potential falling rocks.

7 CASE STUDY 3: HIGH-PERFORMANCE DRAPE MESH SYSTEM

Chain Draping of chain-link mesh over crests and batters of open pit walls is a common practice to hold small rocks back and if they fall out to slow and guide them down to the berm. Under some circumstances standard mild steel chain-link mesh is not strong enough to fulfil these objectives and high-tensile mesh such as TECCO has to be considered.

One of these cases occurred at the Mt Keith Nickel operation North of Leinster in Western Australia. A large berm was developed to protect the ramp underneath it from falling rocks coming from the pit wall above. Concern arose however, as there was one location where a large part of this berm was lost and falling rocks had to be expected to pass it and reach the ramp. Since the ramp is the lifeline of the operation, adequate measures had to be taken.



Figure 15. Location where the berm was lost.

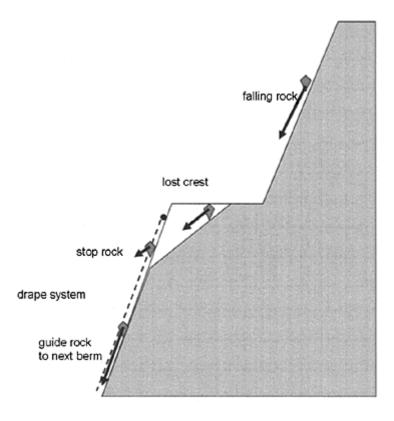


Figure 16. System drawing of the drape system.

In order to cope with this hazard a high performance drape mesh system was designed to stop rolling rocks in the area of the lost berm and to guide these rocks safely down to a catchment area beside the ramp. The system was dimensioned for about 200 kJ and an impact velocity of 10 m/s which equates to maximal block sizes of about 1.5 m³ or 4'000 kg. The mesh rolls are 3 m wide and 30 m long.

Since the drape mesh has to function as a catch fence at its top end, a construction with steel posts and support ropes had to be considered. The 5 m high universal columns were connected to steel tubes which were grouted into 165 mm diameter holes. The length of the grouted-in tubes was 5 m as well due to the ground conditions. The support rope is held at the post



Figure 17. Installed drape system at Mt Keith.

heads and is anchored at both sides to the rock by using twin-strand cables with a loop at the surface. The TECCO mesh rolls are attached to the support rope and connected to each other by using flexible wire rope.

The installation of the drape system was completed in late 2003 and took 5 days with a 3 man crew. Firstly the holes were drilled and then the posts and cable anchors inserted and grouted in. Afterwards the support rope was introduced and attached to the loop anchors with wire rope clamps. Finally the mesh rolls were attached to the rope and released in three stages as lifts were made, developing the batter.

It is important that the mesh does not reach the ground and stops 1 to 2 m above it to avoid accumulating of material at the bottom of the mesh.

This kind of application is a possible solution for situations were standard draping is not strong enough or where mesh is needed with a tested and determined capacity. Examples of such situations are old, badly filled stopes or bad rock conditions with rocks regularly falling out of the batter.

8 CONCLUSIONS

The described case studies prove that flexible rockfall protection barriers and high performance drape mesh systems have to be considered as an option to cope with rockfall hazards in open pit operations.

Rockfall barriers can not only be used to protect local areas from the danger of falling rocks but also as part of pit design optimization. For the first type of application mainly availability and quick installation is important. For the second application such as reducing berm width, double benching or bermless designs, the costs of the system and the saved mining costs (strip ratio) and potential to reach additional ore have to be compared economically.

The current technology allows the design of catch fences with high-tensile mesh for rockfall energies from 20 to 100 kJ and rockfall barriers with ring nets for energies from 250 to 3000 kJ. These systems are engineered and one to one tested.

Consequently, it is possible to dimension sound rockfall bprotection systems by using rockfall simulation software and considering field tested systems and as described in this paper.

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Artificial rehabilitation and control of open pit slope crests and batters

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ABSTRACT: The local stabilities of the crests of ramps and intermediate berms and batters in open pit slopes are critical for production requirements and the safety of operational personnel and equipment. The writers have been involved at several Australian mine sites where partial ramp and berm crest failures have occurred and effective rehabilitation methods not involving pit wall cut backs have had to be developed. The methods are described and evaluated in terms of their effectiveness. At other mines, losses of crests have occurred during mining of benches formed in rock with adversely oriented geological structures. In these cases, pre-reinforcement has been designed and implemented. Prereinforcement has been found to be an effective and economical method of controlling the rock mass behaviour during blasting, maintaining the stability of the crests and providing the design catching capacity of the berms and width of haul ramps. In other cases, rock fall catch fences and curtain mesh have been used to complement the function of berms.

1 INTRODUCTION

In many open pit mines, the presence of structural geological defects, combined with the detrimental effects on rock mass stability of blast vibrations and gases (invasion and heave), results in losses of crests and berms. The changes in bench geometry and loss of rock fall catching capacity compared with the design, in turn, can result in potentially hazardous situations for mine personnel and equipment working at lower elevations within the pit.

In other cases, the crests and berms may be lost from haulage and access ramps. In these situations, the ramp width may be reduced to an unacceptable extent and interruptions to mine production may result.

The writers have been involved in working collaboratively with mine personnel at many open pit operations to develop solutions to these problems. In most cases, several design options are developed. These options attempt to take into account the constraints imposed by the location, available equipment and hardware and the safety to personnel involved in the implementation.

Once a design has been selected, work procedures involving a carefully considered sequence of operations are developed. In some cases this is similar to underground mining operations in which the sequence involves securing potentially unstable ground before moving to the next operation or position.

The methods of rehabilitation and their implementation are outlined as a number of case studies involving open pit mines in Australia. These case studies have involved:

- Rock fall catch fences above underground portals.
- Crest and berm rehabilitation.
- Pre-reinforcement of crests.

In all cases the critical requirement was to provide adequate catching capacity, either in the form of suitably wide berms or via artificial means, should rock falls occur from slope faces above the berms.

2 METHODS OF REHABILITATION

The methods of rehabilitation and protection for operations have included:

- Rock fall catch fences.
- Curtain (drape) mesh.
- Shear pins.
- Cable bolts.

The following types of hardware have been considered and used individually, or in combination, as part of the rehabilitation or pre-reinforcement:

- Flexible woven mesh rolls (mesh curtains).
- Semi-rigid welded mesh sheets.
- Steel pipes, hollow drill rods and solid drill steels.
- Structural sections.
- Steel wire rope and prestressing strand.

3 ROCK FALL CATCH FENCES

The formal design of rock fall catch fences is beyond the scope of this paper but some guidelines have been provided by Pells (1985) and Hoek (2002). It is suffice to say that a formal design involves:

- Identifying the size of potential blocks.
- Identifying the location of the release points.
- Predicting the path of the block after its release.
- Designing a rock fall catch fence with the appropriate geometry and energy absorption capacity

Several computer programs are available to do the calculations of predicted rock fall paths (e.g. Spang & Sonser (1995), Colorado Department of Transportation (2000) and RocScience (2003)). One of the outcomes of the computer simulations will be a range of block sizes and associated velocities of impact with a rock fall catch fence at a particular location. Statistical data on detached block sizes and associated frequencies of occurrence may be collected for natural and permanent excavated slopes. However, data are not available for mine slopes due to the continually changing geometries.

There are manufacturers (e.g. Geobrugg) who are able to design and specify the components of catch fences with various absorption capacities in the range from 25 kJ to 3000 kJ. These catch fences typically comprise:

- Steel posts of various sizes and configurations installed at a specified spacing.
- Steel meshes with different deformabilities.
- Steel wire ropes of different sizes that are attached to the steel posts, interwoven through the mesh and anchored to rock at each end.

The purposes of both the mesh and wire rope are to absorb energy and distribute loading from a potential impact to several of the cantilever beams. The impact of a large block of rock directly to a cantilever beam is likely to cause rupture of the beam (the energy of impact being greater than the estimated energy absorption capacity). The mesh and wire rope fence panels, when loaded laterally, initially deform at low force levels with large displacements. However, the resistance increases as deformations increase. This form of behaviour is required to absorb energy. The resulting deformed profiles of the rope and mesh reduce the lateral forces required to be sustained by the adjacent cantilever beams.

Curtain mesh, comprising flexible woven mesh, is generally rolled down over the batter above a catch fence to cause any detached blocks to remain in contact with the batter during downward movement and to impact on the berm.

3.1 Case study1

3.1.1 Problem description

In this case study, it was required to provide rock fall protection above a decline portal to be developed from an open pit ramp.

3.1.2 Alternative solutions

An option for the major rock fall protection was to construct a rock fall fence on the berm immediately above the portal. However, even with the provision of a mesh curtain installed above the berm it was considered too hazardous to install the fence on the berm. Access was also problematic.

3.1.3 Option adopted

The option finally adopted was to construct a rock fall catch fence (veranda) attached to the face of the slope immediately above and to either side of the portal site as shown schematically in Figure 1. Figure 2. shows a photograph of the completed rock fall fence.

3.2 Case study 2

3.2.1 Problem description

In this case study, several tens of metres of crest and berm losses had occurred in a large open pit.

3.2.2 A lternative solutions

Two alternative solutions were considered:

- Reconstruction of the berms.

- Construction of catch fences.

Reconstruction of the berm would have involved the design shown conceptually in Figure 3.

3.2.3 Option adopted

Since the berm was not required for vehicular access, a rock fall catch fence was adopted. The design was largely based on providing a geometry equivalent to the original berm as shown in Figure 4.

The components of the catch fence are:

- Cantilevers (hollow drill rods) at regular spacing.

- Mesh connected to both the cantilever beams and wire rope.

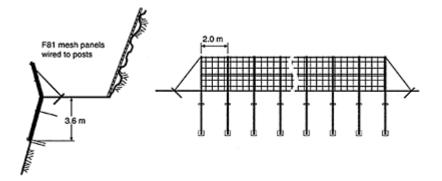


Figure 1. Schematic representation of rock fall catch fence attached to the rock face below the berm level.

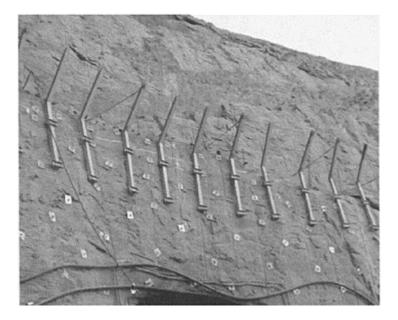


Figure 2. Photograph of the rock fall catch fence attached to the rock face immediately below the berm level.

- Broken rock backfill above the mesh and the rock at the toe of the slope. Wire rope to interconnect the cantilever beams at their extremities.

It was anticipated that it would not be possible to match the lower profile of the mesh with the rock profile. Therefore the mesh was folded up the slope and backfilled with broken rock. In this way, the broken rock provided a cushion for potential impacts close to the base of the mesh and lessened the likelihood of rock bursting through between the mesh and the underlying rock face.

Curtain mesh was draped over the slope faces above the rock fall catch fence in order control the movement of blocks that might detach from the batter and reduce their potential velocity of impact.

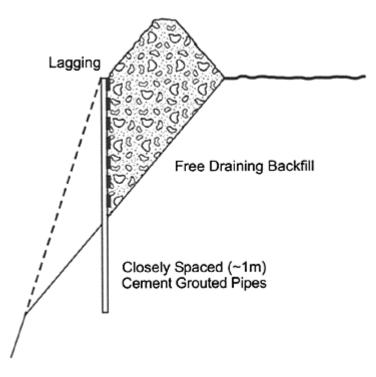


Figure 3. Conceptual design for reconstruction of a berm.

4 CREST AND BERM REHABILITATION

4.1 Case study 3

In this case study a potentially unstable area, involving three large blocks, was identified immediately below the haul ramp of a large open pit.

4.1.1 Problem description

Slope stability in the problem area was required:

- For safety of personnel and equipment working immediately below the blocks.
- To maintain ramp access for the remaining life of the pit.

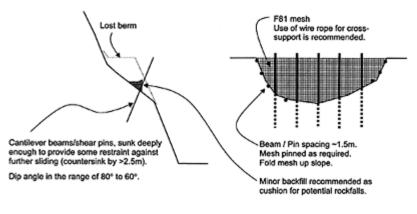


Figure 4. Design of rock fall catch fence with equivalent function to the lost crest and berm.

 To maintain the ramp for access to a portal to a decline for future underground workings.

Observations of crack propagation suggested that there were small, on-going changes to the block configurations. The key issues regarding the likely block stability were:

- Water seeping between the blocks and the supporting plane.
- The steep dip ($\sim 50^\circ$) of the plane on which the main block was supported.
- The stability of the adjacent blocks below and to the side of the main block.
- Future mining activities involving the passage of haul trucks on the ramp and production blasting in the pit immediately below.

The area of seepage suggested that the supporting plane was fully formed. The dip of the supporting plane was in all likelihood greater than the friction angle associated with the plane (estimated to be in the range of 35° to 40°). Without the presence of the adjacent blocks, the stability of the main block was considered to be tenuous.

Previous experience had shown that vibrational energy, particularly due to blasting, may cause small, progressive movements on discontinuities. It was considered likely that the combination of dynamic loading from haul trucks and vibrational loading from tracked plant and nearby production blasts in the pit would similarly cause deterioration in the block stability. Some remedial action was therefore required.

4.1.2 Alternative solutions

In consultation with mine personnel three alternative solutions were considered:

- Block removal by blasting.
- Block stabilisation using reinforcement.
- Combination of reinforcement and partial block removal.

Block removal by blasting would completely remove the hazard with respect to personnel and equipment working in the pit. It was likely that the steeply dipping plane would limit the removal of rock. It was difficult to accurately extrapolate this plane to intersect the ramp but, with the windrow at the crest of the berm, a reduction in ramp width would probably have resulted.

In order to regain the full width of the existing ramp, a retaining structure could (with great difficulty) have been constructed on the steeply dipping plane similarly to that shown in Figure 3.

Stabilisation of the block would be possible provided reinforcement could be installed (conventionally) in the lower region of the block as shown schematically Figure 5. Other locations of the reinforcement would not necessarily address the possibility of further crack propagation and falls of small to medium sized blocks.

4.1.3 Option adopted

The adopted solution to the problem combined aspects of the first two options; that is, a combination of reinforcement and block removal with an associated defined schedule of actions designed to reduce the risks associated with the implementation of the

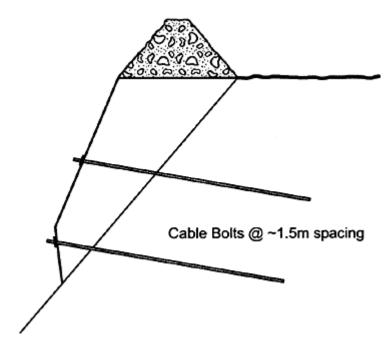


Figure 5. Conventional reinforcement of the potentially unstable blocks using cable bolts.

remedial action. The arrangement of reinforcement and the block geometry resulting after partial block removal are shown schematically in Figure 6.

Figure 7 shows the layout of vertical reinforcement collars. The approximate Factor of Safety against sliding was assessed to be of the order of 2. The perceived advantages of the adopted solution were:

- The ramp width could be maintained (and possibly increased).
- Boreholes could be drilled with equipment located on the stable ramp (behind the steeply dipping plane).
- Ready access to borehole collars for installation of the hollow steel tubes and grouting with cement.
- The hollow steel pipes could be placed immediately after drilling each borehole to provide immediate reinforcement of the block during the drilling of subsequent holes.

4.2 Case study 4

In this case study the crest of a haulage ramp had failed to between 4 m and 5 m behind the crest, resulting in a significant reduction in running width. Potential was identified for further block sliding failure over a similar strike length in the yet to be mined underlying batter.

4.2.1 Problem description

Slope stability in the problem area was required: travelling immediately below the area.

- For safety of personnel and equipment working or
- To maintain ramp access for the remaining life of the pit.
- To prevent further, deeper sliding failures.

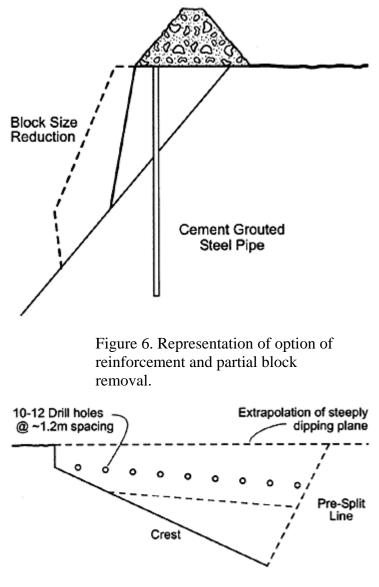


Figure 7. Plan view of approximate layout of shear pin collars.

Mapping data and direct observation of the prevailing structural geological conditions indicated a clear potential for further sliding failures in the affected area. The key issues regarding the maintenance of access and prevention of further failure were to:

- Re-establish the full running width of the ramp.
- Restrain the already disturbed blocks.

- Reinforce and restrain blocks in the lower portion of the batter, which would be exposed by future blasting.
- Provide protection against possible future rock falls.
- Allow future mining activities involving the passage of haul trucks and general mine traffic on the ramp and on a lower portion of the ramp immediately below the area in question.

Remedial actions and precautionary measures were therefore required.

4.2.2 A lternative solutions

In direct consultation with mine personnel two approaches for providing the support required for

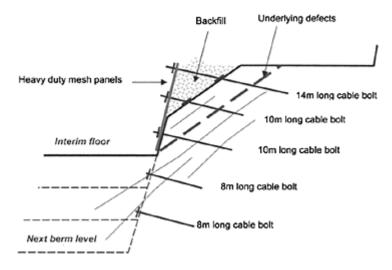


Figure 8. Ramp recovery and batter reinforcement using cable bolts and mesh.

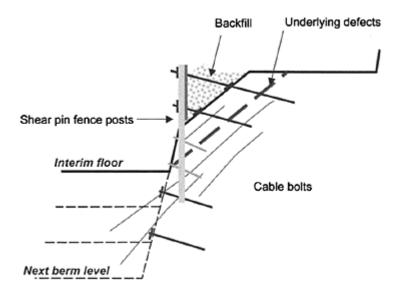


Figure 9. Ramp recovery and batter reinforcement using shear pin fence and cable bolts.

re-establishment of the ramp and deeper reinforcement against possible future sliding failure were considered:

– Cable bolts and mesh (Figure 8).

- Shear pin fence and cable bolts (Figure 9).

4.2.3 Option adopted

The adopted solution to the problem was dominantly based on the cable bolt option, but also incorporated shear pins to reinforce the interim toe of the batter. The arrangement of deeper reinforcement and the ramp crest and batter geometry are shown schematically in Figure 10. Cable bolts were adopted because, with the use of a boom-mounted drill rig, reinforcement could be installed more uniformly throughout the mass containing the defects on which future sliding could potentially occur. Access to install shear pins was severely limited. Cable bolts would also have been needed to restrain the cantilevered ends of the shear pins.

Installation of reinforcement and support elements had to be carefully sequenced in order to ensure that

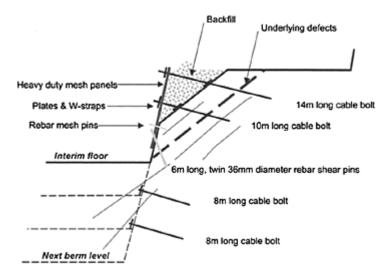


Figure 10. Ramp recovery and batter reinforcement using cable bolts and shear pins.



Figure 11. Photograph of as installed ramp remediation.

operators were always within a safe working area. The cable bolts were installed and lightly post-tensioned against the *in situ* rock, leaving an appropriate free length to hold

the mesh in place. Backfill was placed after the mesh was fully secured. The successfully completed remediation is shown in Figure 11.

5 CREST PRE-REINFORCEMENT

The following two case studies address the problems caused by the loss of crests due to adversely dipping geological structures and blast damage. Visual observations after blasting suggested that blocks were destabilised due to blast gases causing upward directed forces and loosening of blocks near the crests (blast-induced block heave). Subsequent mechanical scaling removed these loosened blocks. In order to maintain the crests, it is necessary to pre-reinforce these potentially unstable blocks before blasting using steel elements grouted with cement installed in sub vertical boreholes.

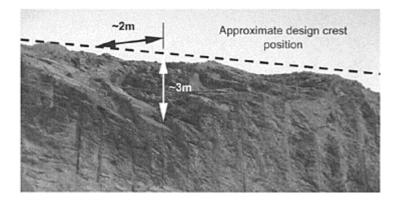


Figure 12. Photograph showing loss of crest rock relative to approximate design crest position.

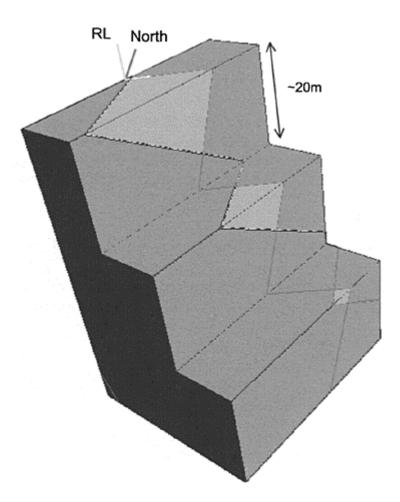


Figure 13. Computer generated graphical representation of observed potentially unstable block shapes and sizes (~20 m bench height).

5.1 Case study 5

5.1.1 Problem description

In this case study it was estimated that approximately 25% to 30% of crests were damaged due to the ubiquitous presence of adversely oriented arrangements of joint sets. In most cases, the loss of crest was restricted to a few metres both horizontally and vertically as shown in Figure 12. However, occasionally, the potentially unstable block

could affect up to ~10 metres behind and below the intended crest position with a strike length of up to 20 metres. A computer simulation of the potential size of observed blocks is shown in Figure 13.

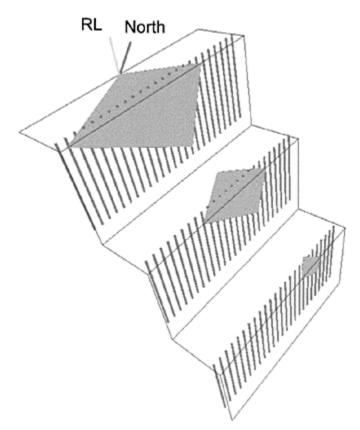


Figure 14. Computer generated graphical representation of arrangement of potentially unstable blocks and vertical shear pins.

5.1.2 A lternative solutions

Two possible solutions involving the use of various different sizes of "shear pins" were considered:

- Large diameter, thin-walled steel pipe.

- Smaller diameter thick-wall steel pipe.

The arrangement of shear pins for the range of potentially unstable blocks sizes is shown in Figures 14 and 15.

5.1.3 Preferred option

The preferred option to use large diameter, thin walled steel pipe was based on experience gained in underground mining at Broken Hill where long (55 m), steel pipes were installed parallel to the planned open stope wall position and were successful in stabilising the blocky rock mass. At that time, it was assessed that the use of higher strength, but much lower stiffness and more flexible steel strand would not have been as successful as the pipes. In some cases, the stope wall was undercut locally and the strand would have buckled invariably whereas the pipes were more stable in compression and bending.

In some trial installations at this mine, the preferred option was not adopted and was replaced by the use of solid drills steels grouted with cement in smaller diameter boreholes. These trials were partially successful in reducing the crest break back.

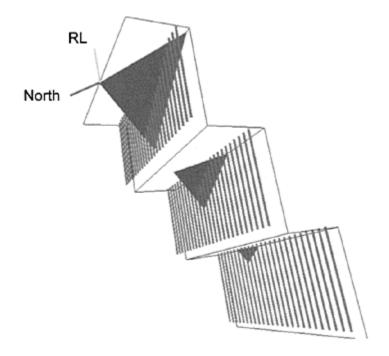


Figure 15. Computer generated graphical representation of interaction between shear pins and large (predicted) unstable blocks (looking upward from within the rock mass).

5.2 Case study 6

5.2.1 Problem description

In this case study, similar problems to those described in the previous case study were anticipated.

5.2.2 A lternative solutions

Similar alternative solutions to those described in the previous case study were evaluated; namely, different size steel pipes, hollow drill rods and solid drill steels.

5.2.3 Option adopted

In this case, ~90 mm outside diameter by 5 mm wall thickness galvanised steel pipes were adopted. The ~6.5 m long pipes were installed in ~102 mm diameter boreholes drilled approximately vertically about 1 m from the design crest position. Some difficulties were experienced during placement of the pipes due to borehole deviation and the relatively small clearance between the pipe and borehole.

5.2.4 Performance assessment

The performance of the shear pins was deemed to be successful. Figure 16 shows an example of a large otherwise unstable wedge that has been retained with minor movement from its original position.

6 CONCLUDING REMARKS

The rehabilitation methods developed and described are usually not amenable to formal design procedures. The methods and specifications have evolved by using engineering judgement and working within the limitations imposed by the available materials and equipment to be used for installation and accessibility to complete the work safely.

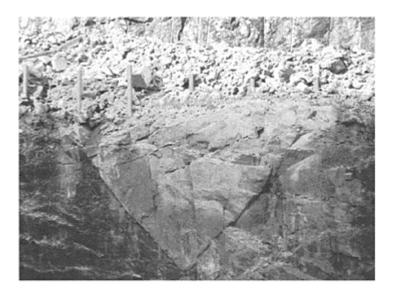


Figure 16. Photograph showing the effectiveness of shear pins in stabilising a large (otherwise unstable) wedge.

The methods for crest rehabilitation are only applicable to containment of bench scale failures. It has been found that pre-reinforcement of crests is an economical solution in reducing, and possibly eliminating the requirements for rehabilitation.

ACKNOWLEDGEMENTS

The writers wish to acknowledge the assistance and cooperation of mine personnel in the development and implementation of the various rehabilitation and protection measures.

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6 Dynamic testing

Dynamic capable ground support development and application

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ABSTRACT: The rapid fracturing and deformation that occurs around excavations during rock bursts demand that support and reinforcement systems have both deformability and damping capabilities. The energy demand and supply approach currently used in design is considered to be too simplified to represent adequately the rock burst process and the support and reinforcement system requirements. Momentum change may be a necessary second criterion for the analysis of dynamic loading situations. Based on the improved design rationale, the St Ives Gold Mining Company has developed and implemented dynamic capable ground support systems. The systems consist of mainly the DE (Dynamic capable and Environmentally friendly) plates made from conveyor belt rubber, and the Dynamic Cables with sliding anchors, as well as the process and procedures for their selection and application. The implementation and monitoring of the systems since 2002 have demonstrated that the systems have adequately maintained integrity and serviceability of excavations under severe rock bursts and seismicity.

1 INTRODUCTION

Alleviating the rock burst hazard is important to the safe and economic operation of many underground metalliferous mines. The most commonly used approach to the design of dynamically capable support and reinforcement systems for underground rock burst conditions is based on energy considerations. In the energy approach, it is postulated that the reinforcement elements should possess yielding capability for a specified deformation, velocity and displacement. This has led to an emphasis being placed on the development of yielding reinforcing elements, although attention has also been paid recently to improved surface support techniques such as sprayed liners.

In recent years, the Geotechnical Team at St Ives Gold Mining Company (SIG), Gold Fields Ltd (formerly WMC) has questioned the existing design methodology and systems. An alternative approach has been developed which is based on momentum change and emphasises the importance of the damping capability of the support and reinforcement system. As part of this effort, new products, including what are referred to as DE plates and a new dynamic cable bolt, have been developed and are being used at SIG and a few other Western Australian mines. The fundamentals of the development of the dynamic support system were discussed in a previous paper (Li et al. 2003). Some testing results, the system element details, and the actual performance are discussed in the present paper.

The terms dynamic support and dynamic support systems will refer to all surface support elements, surface support fixtures, and reinforcing elements. Surface support elements refer to mainly shotcrete, mesh and straps. Surface support fixtures refer to plates, wedges and barrels, nuts, Split Set rings, and other connection elements. Reinforcing elements are rock bolts, tendons and cables. Although, strictly, the total system should be referred to as the support and reinforcement system, in the interest of brevity it will be referred to here as the support system.

2 CURRENT APPROACH

2.1 Rock burst mechanisms

A rock burst has been defined as the uncontrolled disruption of rock associated with the violent release of energy additional to that derived from falling rock fragments. By definition, a rock burst causes damage to the rock around an excavation. Rock bursts are a sub-set of a broader range of seismic events that are associated with unstable equilibrium on the mine global or more local scales.

It is widely accepted that there are two modes of rock mass deformation or response which lead to instability, mine seismicity and rock bursts—slip on natural or mininginduced planes of weakness, and fracture of the intact rock itself, usually close to the excavation boundaries. In either case, excess energy will be released from around the source of the instability and propagate through the rock mass as a series of seismic waves. These waves will induce dynamic stresses and associated displacements within the rock mass. As well as the usual body (compression and shear) waves, surface waves may result near excavation boundaries. Waves may be refracted and reflected at interfaces and boundaries of various kinds.

2.2 Rock burst support design theory

An energy absorption theory is generally used for the design of support systems for rock burst prone conditions. It is postulated that the damaged rock mass around an excavation releases a certain amount of energy and that the support system must be capable of absorbing this energy. In its simplest form, this energy is calculated as

 $E=\frac{1}{2}m v^2 + m g h$

where E is total energy of the rock ejected from an excavation, m is the mass of the rock, v is the ejection velocity, and h is the height fallen through, or displacement, of the ejected mass of rock.

The support system capacity is often represented as a force or resistance generated, F, and a displacement capacity, d. For the support system to be effective, the following must be satisfied:

F d \geq ¹/₂m v²+m g h

(2)

(1)

One of the shortcomings of this theory is its inadequacy in considering the cyclic, oscillating, and/or hysteretic nature of the dynamic loading conditions recorded through monitoring (Cichowicz et al. 2000, Cichowicz 2001). Another difficulty is the selection of an appropriate ejection velocity. Typically, design velocities have been between 3 and 10 m/s, with a recent trend towards lower values. Sometimes the peak particle velocity (ppv) of the rock mass is used in place of the ejection velocity.

An outcome of the application of this approach has been the requirement that a reinforcing element must be capable of yielding to a designated displacement, d, for a given bolt or cable force, F. The underlying assumptions and requirements are: (i) the rock mass within the supported area moves in unison at the same velocity, v; (ii) this rock mass velocity is reduced to zero by the support system after a displacement, d; and (iii) all support system elements, including the reinforcing elements and surface fixtures, are compatible, with the "strength" of the weakest element being equal to or greater than F. However, field observations of rock mass damage and system performance suggest that some, and often all, of these assumptions and requirements may not be satisfied.

2.3 Rock burst and support system behaviour

The rock mass damage processes operating during a rock burst may include new fracture generation, extension of pre-existing fractures, and irreversible deformation on preexisting discontinuities or new fractures. The fracturing associated with a rock or strain burst often results in fine fragmentation of the rock mass, possibly within a fraction of a second (Ortlepp 1997). Generally, the rapidly disintegrating and moving rock pieces may not "see" the installed bolts or/and cables. There are ample observations of rock burst damage (Ortlepp 1997) in which very few bolts or cables were broken. In these cases, the bolts or cables may appear to be extruded from the damaged excavation surfaces as shown in Figure 1. In this



Figure 1. A strain burst with very little damage to bolts and cables (Li and Singh 2000).

case, several tonnes of rock fell through broken mesh during a minor strain burst, but with very little damage to the bolts and cables.

Observations made by Haile (1999) in South Africa showed that relatively stiff rebar rock bolts, having very limited yielding and energy absorption capacity, could survive major dynamic deformation and damage. It was suggested that energy was being dissipated in deformation of the rock mass which was contained by mesh and lacing.

3 AN ALTERNATIVE DYNAMIC SUPPORT RATIONALE

In 1998, SIG faced a demand for viable dynamic capable support systems to combat escalating seismicity and rock bursting. The SIG Geotechnical Team embarked on developing an improved understanding of seismicity and mining method/sequence, which resulted eventually in an underhand open stoping with paste fill (Li & Singh 2000, Li et al. 2002). Some enabling techniques were developed, such as dry tailings made paste fill and the alternative rationale for rock bursts and dynamic capable ground support systems as part of an overall risk management strategy for seismically active mining (Li et al.

2002a). As a result, a multi-tiered ground support regime was developed and implemented.

3.1 Rock burst process and demand on support

The failure or damage modes in a discontinuous rock mass arise typically from a combination of shear and tension. Fracturing of otherwise intact rock may also occur. Shear is more likely the dominant further into

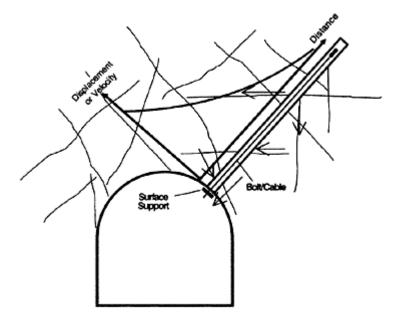


Figure 2. Potential displacement modes and demands on system deformation, velocity and acceleration (and deceleration).

the rock mass than at the excavation surface. When the support system is mobilized, the reinforcement elements are more likely to be subjected to shearing at depth, while the surface support elements are more likely to be subjected to tensile loading and buckling (Figure 1). These shear and tensile loading mechanisms may be more pronounced in large block ejection rock burst events than in the fine fragment ejection type.

In case studies of rock burst damage and support performance in South African mines, Güler et al. (2001) observed shear failures of rock bolts at 70% of the sites surveyed. The deformation profile at the excavation surface might be expected to be different from that further into the rock mass (Figure 2). If the rock mass deformation is characterised by displacement, velocity and acceleration (or deceleration), the closer to the excavation surface, the greater will be the magnitudes of these parameters and possibly the greater their rates of change. It is logical to assume that the demands on dynamic support systems will follow similar trends. Ortlepp and Stacey (1998) found that it was often inadequate terminating arrangements on the reinforcement that failed prematurely under dynamic conditions.

The magnitudes of the seismic events and of the induced deformations also have important implications for the design of dynamic support systems. The failure mode near the excavation surface in a rock burst event is generally dominated by intense fracturing and disintegration of the rock mass (Ortlepp 1997). The size of the failed rock mass and its interactions with the support system will have a major influence on support system selection and design.

3.2 Dynamic loading from seismic waves

Despite the attention that has been paid to rock bursts and their effects over the last 40 years, the nature of rock burst waveforms and the associated dynamic loading have not been studied as intensively as might be expected. There are, of course, some notable exceptions to this general observation. Generally, seismic events in mines may be characterised by parameters developed originally in earthquake seismometry. Source parameters such as energy output, seismic moment, source radius and radiated energy may be related to the damage sustained by mine excavations. Despite the similarities between earthquake and mining seismicity, the design methods used in earthquake engineering have not been applied to the design of mining support systems.

The interactions between seismic waves and excavation surfaces are very complex, depending on the wave frequencies. Very low frequency waves are less likely to cause damage than high frequency waves. From seismic wave energy theory, it can be expected

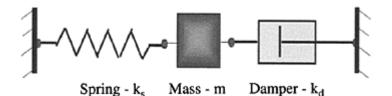


Figure 3. Simple schematic representation of a loadingsupport system.

that incipient seismic waves may change phase, magnitude (often increasing), and type (from compression or shear waves into tensile) at rock/air interfaces. The peak particle velocities (ppv) at excavation surfaces can be magnified 4 to 10 times compared to those measured within the rock mass (Cichowicz 2001).

The typical representation of seismic loading used in earthquake and civil engineering uses waveform characteristics as input, and gives forces, displacements and displacement rates as output. For the simple system shown in Figure 3, the following theoretical representation can be made:

m a+v k_d +d k_s =F cos (ω t)

(3)

where k_d is the damping coefficient of the system, k_s is the stiffness of the system, ω is the frequency of the loading waves, and F and d are the force and displacement, respectively.

An important difference between dampers (k_d) and springs (k_s) is that springs (ideally) store all the energy as potential energy, while dampers (ideally) dissipate all the energy as heat. Energy dissipation ability is proportional to the damping coefficient. The dynamic and cyclic seismic loading associated with rock burst events has important practical implications. Measurements made by Cichowicz et al. (2000) showed that significant amounts of energy can be reflected back into the rock mass during a seismic event. Magnification of signals can occur near the surface of an excavation, and fractured rock surrounding an excavation can cause scattering of seismic waves. Cichowicz et al. (2000) concluded that the complex rock mass around an excavation should be studied using a multiple degree of freedom model. The cyclic nature of seismic ground motion should be allowed for in modelling and design.

3.3 Momentum change

The preceding discussion shows that the dynamic loading of the rock mass and support system in a seismic or rock burst event is a very complex process. From a mechanistic perspective, there is an initial acceleration of the rock mass induced by the stress waves. This will impose dynamic loading on the surface support elements and fixtures as well as on the reinforcing elements. At some point, the accelerated rock mass and support system will reach their maximum velocities (which may or may not be the same for the rock mass and the support system elements). To fully mobilise the support system capacity and to maintain the integrity of the rock mass-support system, the rock mass and support system must decelerate from the peak ejection velocity and come to rest at a limited displacement over a short period of time.

The ability of the surface support to accommodate these sudden changes in velocity is of vital importance to the effective dynamic performance of the system. Momentum change theory can be useful in establishing the requirements in this regard. Rinehart (1975) observes that "momentum is similar to energy in that it cannot be destroyed but it has the added advantageous quality that it cannot change its identity and can be kept track of easily." The change in momentum of the system is

 $\delta M = mv_1 - mv_2$

(4)

where m is the total mass of the system, and v_1 and v_2 are the initial and final instantaneous velocities. This assumes that the rock mass and the support system have a constant mass. However, the disintegration of the rock mass during a rock burst event suggests that the rock fragments may have differing velocities.

The momentum change is related to the resisting force applied over a period of time, t, through:

F t=
$$\delta$$
M or F t=m δ v

This is, in fact, the impulse loading, which can also be expressed as

F=m a

(6)

(5)

where a is the acceleration (or deceleration) and F is the force.

When momentum change as expressed by Equation 3 is considered in the design of a dynamic capable support system, it introduces an important second criterion to be satisfied in addition to the energy absorption criterion as expressed by Equations 1 and 2.

3.4 Rock mass damping capacity

A rock mass has often been described simply as a linear elastic medium. However, a more realistic description is that a rock mass possesses both elastic and plastic characteristics and a rock mass may exhibit both elastic and plastic behaviour depending on the loading status and the integrity of the rock mass.

The recognition of the importance of system damping on the system load bearing capacity or system survival capacity, opens a new avenue, perhaps an important one, for dynamic capable support system design and engineering.

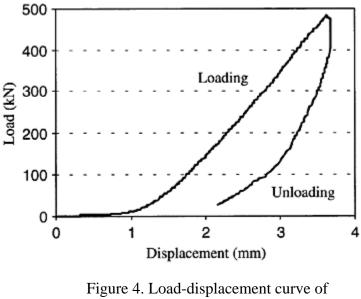
The use of DE plates at Junction and a few other mines is in fact an effective way to improve the system damping capacity and hence to improve the system load bearing capability. The DE plates, combined with steel plates, serve to reduce the impact loading through reduced effective velocities imparted to the steel support and reinforcement system, similarly to the processes analysed by Zhan et al. (2000) for a vehicle suspension system.

4 DYNAMIC SUPPORT SYSTEMS

4.1 Dynamic support system-rock mass interaction

The overall system is taken to consist of the rock mass surrounding the reinforcing elements plus the reinforcing and surface support elements and fixtures. The mass of the loading system, m, can be assumed to be the total mass of the n fragments contained, or Σm_i . The mass of the support system is assumed to be negligible by comparison. The displacement of this rock mass is constrained by the designated displacement, d. The surface support elements will have to sustain the same amount of displacement, d. However, the reinforcement elements may or may not be required to undergo the same displacement. It is assumed that the reinforcement elements provide the constraining force required, F. The energy-based design approach logically suggests that the dynamic capacity of the system can be increased by increasing the designated displacement. Accordingly, there has been an emphasis on developing yielding bolts and cables for this purpose.

The stiffness, k_s , and damping coefficient, k_d , of the rock mass-support system are very difficult to determine. If the rock mass is regarded as an assembly of rock fragments, its stiffness should be comparatively small and could be ignored. The stiffness of the whole system can then be simplified as the stiffness of the support system. As suggested by Equation 3, the damping coefficient has a direct impact on system velocity and velocity change. Damping influences momentum change. In a rock mass-support system, damping may exist, or can be introduced, within the rock mass, as well as at the interfaces between the rock mass and the reinforcing and support elements. Damping may also result from the friction between fragments generated during motion. The damping coefficient of the overall system can be approximated as that of the support system. This will generally underestimate the total damping capacity of the rock masssupport system. The rock mass itself may have considerable damping capacity



DE plate.

associated with joints and faults and stress- or blast-induced fracturing.

4.2 Dynamic support elements and systems

As part of its investigations of dynamic capable support systems, the SIG Geotechnical Team carried out a wide range of laboratory tests, in situ tests and trials. The simple tests carried out on support elements demonstrated that there are often mismatches between surface fixtures, surface elements and reinforcing elements. As a result, plates made from conveyor belt rubber (subsequently called DE plates, with DE standing for Dynamic capable and Environmentally friendly) were trialled, proven and eventually used to improve system dynamic capability (Li 2001).

The compression test loading-unloading curve shown in Figure 4 illustrates the energy absorption capacity of an 11mm thick DE plate. DE plates improve system damping capacity and hence system load bearing capability. When combined with steel plates, DE plates reduce impact loading by reducing the effective velocities imparted to the steel support and reinforcement system.

Simple drop weight tests were performed for the combinations of different plates and washers with and without DE plates. These tests generally showed much less damage to the steel plates when DE plates were used. Yi and Kaiser (1994) demonstrated in earlier laboratory tests the effectiveness of rubber and soft wood "shock absorbers" in a support system. The analysis of their laboratory scale tests was based on energy theory. The steel bars used were assumed to have constant stiffness under dynamic loading. In fact, the yield strengths and moduli of most steel alloys increase with strain rate. Fracture resistance generally does not increase to the same extent as yield strength, so the likelihood of tensile fracture can be expected to increase with strain rate.

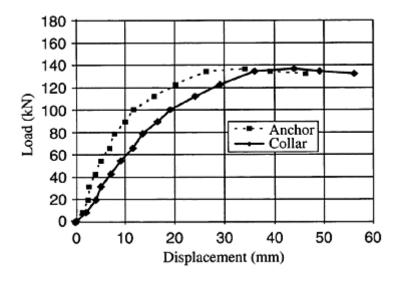


Figure 5. In situ pull test results of D cables.

This is very significant in the current context, suggesting that even at moderate loading rates, the increased stiffness of a solid bar during seismic loading could result in failure of the bar, rather than mobilising sliding or "yielding" mechanisms. Ortlepp (1997) observed that a grouted rebar failed with a "chalk-stick" appearance and no "necking". Similar abrupt failure modes have also been observed for de-bonded bars in seismically active mines in Australia.

An underlying cause of these failures may be the mismatch between the low and reducing stiffness of the rock mass during a rock burst and the high and increasing stiffness of the reinforcing bar. In an undamped support system, the high input velocities from seismic waves and the rapid change in velocity could result in high impulse loading which could "snap" the bars. Using this rationale, a high capacity yielding cable rather than a rebar-based reinforcing element, known as the Dynamic Cable (D Cable), was developed in conjunction with Garford Ltd. Extensive static loading and dynamic loading tests as well as in situ pull tests and trials have demonstrated their ability to respond to both slow loading and very rapid loading up to 3 m/s. Figure 5 shows typical underground pull test results. These cables are now used as the main dynamic reinforcing elements in SIG mines.

4.3 Dynamic support system characteristics

It is concluded that a dynamic capable support system for rock burst conditions should possess the following characteristics:

- the system should be able to withstand both shear and tensile loading;
- the system should have a limited deformation dictated by the allowable serviceability of the excavation, but allowing greater deformation at the excavation surface;
- the system should be able to not only absorb seismic energy, but also to withstand momentum change or to reduce impact loading by the rock mass; and
- the system should be able to survive multiple seismic events.

A holistic approach to the design and engineering of a dynamic support system should consider the stiffness, damping capacities, and repetitive and dynamic load bearing capacities of the elements, as well as the selfsupporting characteristics of the rock mass. A simple first step is to improve the key capacities of the system's "weakest link" which is often the surface fastening elements.

4.4 Multi-tiered support system

At SIG's Junction Mine, two ground support and reinforcement strategies have been developed; one for low stress non-dynamic conditions and one for high stress dynamic conditions.

The primary support system used in low stress, nondynamic conditions consists of 2.4 m long, 46 mm diameter grouted friction bolts and 5.6 mm diameter wire weld mesh. All components are galvanised. This system proved to be inadequate for the dynamic conditions experienced at depth. The behaviour of the grouted friction bolts was erratic with some yielding and others either breaking or losing their rings. Grouting of friction bolts was discontinued. This was adequate for low to medium level strain bursts in development headings, but was not adequate when stope extraction commenced or for the increase in seismicity experienced with increasing depth. Secondary support and dynamic support had to be installed some distance, often more than 40 m, from the extraction stope brows. As shown in Table 1, four levels of dynamic support have been developed and implemented.

In the primary dynamic support cycle, the heading is not scaled, although obvious loose rocks are knocked off. The heading and face are shotcreted in cycle with 50 mm thick fibre reinforced shotcrete applied to within 1.5 m of the floor. The heading is then meshed and bolted over the shotcrete as in the non-dynamic case. For dynamic conditions, the friction bolts are not grouted and DE plates are used with the bolts. The

DE plates reduce damage to the friction bolt rings during installation, reduce damage to the mesh, and act as damping elements during rock bursts.

Not scaling the headings has had a significant impact on the levels of seismicity experienced in those locations. It is believed that the shotcrete seals in the cracked and unscaled rock around the excavation resulting in a low stiffness surface layer which provides several benefits. In particular, the undamaged rock interface is pushed away from the surface

Multi-tiered dynamic ground support			
Standard	Primary	Mesh over shotcrete+ungrouted friction bolts with DE plates	
	Secondary	Primary+straps+D cables+DE plates	
Elevated		Primary+straps on a cross pattern+D cables at a higher density+DE plates	
High		Yielding sets	

Table 1. Dynamic support levels, junction mine.

and is cushioned by a layer of damaged rock which has a lower stiffness and provides an interface at which the seismic wave is reflected, refracted and attenuated. The shotcrete also contributes to these effects and hampers the propagation of damaging surface waves.

The primary dynamic support system proved adequate for development headings. A secondary dynamic support system was developed to improve the system's containment and retainment capabilities when stoping progressed. This secondary system now consists of the primary dynamic system and the following additional elements (detailed in Table 2):

- 6 m long D cable bolts on 1.5 m spacings and approximately 2 m ring spacings to coincide with overlaps of the mesh;
- DE plates on the bolts to provide damping; and
- mesh straps at the yielding cable bolt rings to tie in the cables and provide reinforcement to the mesh.

Where high seismic and rock burst hazards are anticipated, the elevated dynamic support system is used. This level is similar to the secondary level except that the yielding bolt and ring spacings are 1.2 m to 1.5 m. Where larger events or severe rock bursts are expected, the high level dynamic support used consists of yielding arches with cemented fill injected between the arches and the rock.

4.5 Performance of dynamic support systems

The implementation of the dynamic capable support systems was carried out in two areas: rehabilitation of the existing production levels and primary and secondary support for the future production levels. Use of the dynamic support also extended to seismic and rock burst prone areas, such as fault intersections and drives/cross-cuts near pillars.

In one case, in which the elevated dynamic support system survived a large rock burst, of approximately 1.5–2.0 local magnitude, the drive convergence was up to 0.7 m over a 20 m length of the drive from the stope brow. However, the fragmented rock around the

excavation was fully contained (Figure 6). Evidence suggested that most, if not all, of the D cables slid with the rock mass during the burst. Examination of the

Ground support elements			
Element	Specification		
Shotcrete	50 mm minimum, thickness		
	75 mm maximum, thickness		
	MPa 28 day strength		
	onthetic fibre, 7 kg per m ³ of BARCHIP,		
	50 mm long		
Weld mesh	Pre-galvanised, 5.6 mm wire on a 100 mm×100 mm grid (BHP G445)		
Chain-link	Currently being evaluated (GeoBrugg mesh high tensile)		
Straps	Mesh straps: 5.6 mm wire, 3×7 mm longitudinal strand		
Friction bolt	SS46 or equivalent, 2.4 m long, galvanised		
	SS39 or equivalent stubby bolt for pining mesh overlap, 0.6 m long galvanised		
Plates	Galvanised dome plate for bolt (150 mm×150 mm×4 mm) D Plate 300 mm circular(rubber backing plate) Washer and butterfly plate 400 mm× 280 mm×1.9 mm		
D Cables	Garford dynamic cables; 300 mm travel and 14 t Slip rating, with end stopper. To be used with Garford plates, barrel and wedges.		
Grout	Low heat cement, potable water; W:C ratio=0.3 to 0.35		

Table 2. Summary of ground support elements.

recovered D cables indicated that a few cables had snapped at the stop lock after the sliding anchors had slid the full design length of 300 mm.

Recent observations also illustrate the contrasting performance of dynamic support and the conventional support. In a moderate rock burst, severe cracking of shotcrete and minor movement occurred when elevated dynamic support systems were installed, but ejection of up to 0.5 m of rock mass occurred where no dynamic support was installed (Figure 7).

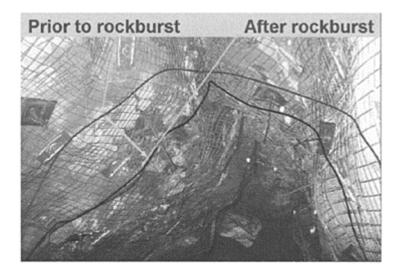


Figure 6. Drive profiles before and after a large rock burst where the broken rock mass was contained by the dynamic support systems.



Figure 7. Contrasting performance of dynamic and nondynamic systems during a moderate rock burst.

5 CONCLUSIONS

The new SIG approach to analysing the rock burst damage process and the demands placed on dynamic support systems, emphasises the nature of seismic loading, momentum change, the reflection and refraction of seismic waves, rock mass stiffness, support and reinforcement stiffness and support damping capacity.

This approach has led to the development of two major new products, DE plates and D cables, which have been implemented as part of a multi-tiered set of dynamic support systems. The dynamic support systems have demonstrated their capability of surviving moderate to large rock bursts over the last two years in a seismically active mine.

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Field performance of cone bolts at Big Bell mine

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ABSTRACT: Big Bell introduced cone bolts in 1999 as one of many ground reinforcement options investigated to manage the damage resulting from rockbursts. The timing and location of installation of the cone bolts changed with mine requirements from 1999 to 2002. Initially the bolts were installed in the footwall amphibolite access development, before the development was exposed to stoping stress change. Following their success in maintaining rockbursts the bolts were installed in oredrive schist. The bolt was then exposed to the higher deformation ground response from the schists. The bolts were then exposed to the complete deformation history of the drive when installed with the development cycle. The paper will discuss the factors for cone bolt breakage, the required modification to installation practices, and improvements introduced to maintain the entire support system performance.

1 INTRODUCTION

This paper will specifically address the performance and installation requirements for the cone bolts in the Big Bell environment. The history of rockburst activity, changes in the understanding of rockburst conditions, the sublevel caving process and geotechnical environment at Big Bell has been fully described by the author in other papers. These are recommended for additional information.

The cone bolts were first installed in the competent footwall access development. The development was prone to rockburst events during the production cycle in 1999. The foliated footwall amphibolite had a UCS range of 70–200 MPa, mean of 126 MPa and Young's Modulus of 66 GPa. Observation of the footwall drives and extensometer monitoring indicated minor displacements in the backs and walls.

Essentially minor displacements occurred prior to a large dynamic displacement. In this environment the cone bolts proved to be relatively insensitive to installation practices. Cone bolts were introduced to oredrives within high stress areas, following a severely damaging oredrive event.

Mining of the lower levels of the mine were suspended in 2000 following a rockfall fatality. A mine planning phase followed with associated redevelopment of the lower

levels and installation of an improved ground support system to withstand the damage from rockbursts.

Mining recommenced in 2001 utilising cone bolts in development defined as having a seismic potential. Development cross cutting the foliation performed well with minor observed deformation, primarily due to the orientation of the drive to structure and stress field.

In the ore strike drives a problem was encountered with the head of the cone bolt breaking off. The occurrence of cone bolt breakage was initially considered to be a result of developing in highly stressed and deformable ground, without an appropriate modification to the installation practices. This was found to be partially accurate; however the steel properties of the bolt were also important.

Failure of the cone bolt was not common to all strike development. It occurred primarily in the backs and shoulders of the footwall oredrives, affecting an estimated 1% to 10% of bolts in an area. The overall number was less than 0.1%. The cone bolt was the eight different reinforcing elements to be installed at Big Bell and break. Prior bolts included the: Hollow Groutable Bolts, Solid Point Anchor Bolts, two styles of friction stabilisors (Splitsets and Hardie Bolts), Tubular Groutable Bolts, Gewi bars, and Cable Bolts.

The situation of cone bolt breakage was not a case for the abandonment of the bolts, but rather to develop a better engineering understanding of the factors affecting the performance of the bolt.

1.1 Initial dynamic performance

The initial cone bolts were 3.0 m in length. They were installed in a low heat thick mix grout, of average 28 day cube testing strength of 52 MPa. This can be related to a 2:1 cylinder UCS test strength of 40–45 MPa from Rong et al 2002. The performance of the cone bolts under dynamic loading conditions was good, with strong deformation of the original dome plates.

The damaging seismic events of 1999 and 2000 that overwhelmed the installed reinforcing elements, did not necessarily allow an assessment of the integration of the reinforcement and support systems. Only events that did not fail the reinforcement elements allowed an assessment of the integration of support and reinforcement elements. Once integration of support and reinforcement elements was realised there would also be a redundancy component built into the ground support system design.

1.2 Loading mechanisms of rock bursts

It was probable that two rockburst mechanisms existed at Big Bell and could be asssessed from differing loading of the support system. The rockbursts resulted in either;

- 1. a 'simple' closure of the drive with little bulking of the mesh, displacement of the semi-intact ground, nearly all the resistance work is done by the reinforcement, or
- heavy loading of the surface support and general loading of reinforcing elements. This can lead to the complete destruction of the surface support or surface attachment elements, without necessarily destroying the reinforcing elements although they could.

Occurrence of general closure of the backs and walls with minor bulking, probably related to events remote from the excavation and was possibly represented by a Peak Particle Velocity loading of fractured ground or a stressed structure. In this situation the seismic source was remote from the damage.

The complete failure of the mesh (with occasional failure of the reinforcement elements, Figure 1) was probably from a 'high skin' velocity rockburst. The seismic event source style would be considered as very close to the excavation (within one excavation diameter?) with shear rupture on a foliation plane structure, or strain burst of highly stressed intact ground in the back of the excavation. In these cases the seismic source of the event is at the damage location. Shear rupture on the foliation plane has the potential to fail any reinforcing element the crosses the plane. But its support and control has been attempted and was successful.

When a tougher weld strength surface support mesh was introduced then the tearing of the mesh welds was no longer observed, in rock bursts. An example of bulked mesh is shown in Figure 2. The weld mesh properties used are in Table 1. The standard for seismic support mesh at Big Bell was defined as the M71 product, prior to it was the RF81 product.

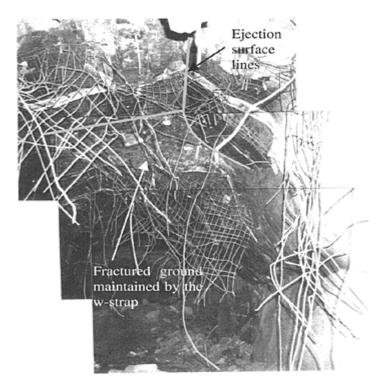


Figure 1. Shredded mesh from February 1999 rockburst.

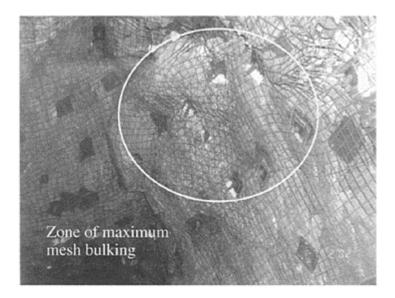


Figure 2. Bulked mesh damage held.

Туре	Wire diam. (mm)	Weld shear strength (kN)	Wire yield strength (Mpa)	Wire tensile strength (kN)
Industry	5.3	7	380 Yield	8.4 Yield
Galv weld mesh			550 Failure	12.1 Failure
M61	6.3	12.5	450	14
M71	7.1	15.8	450	17.8
RF81	7.6	11.3	500	22.6

Table 1. Weld mesh properties.

2 TEST WORK

Test work commenced in 2001 on a number of parameters considered to be key for the use of cone bolts:

-bond strength on the smooth shaft of the bolt,

-rapid set grout strength for early tensioning,

-pull out capacity of cone bolts,

-strength testing of grout mixes.

2.1 Smooth bar bond tests

Cone bolts were required to be order in large quantities from South Africa. With an irregular delivery schedule a large number of bolts were on site at the time of mining suspension in 2000. At the recommencement of installing cone bolts approximately 10 months latter, it was noted that some bolts exhibited a substantial deterioration or flaking of the protective wax coating. A decision was made to test the performance of the coating, in debonding the bolt from the grout. Results from the tests are shown in Figure 3 and Figure 4.

Jager et al 1990 published one of the first papers on cone bolts. It stated that the cone bolt should have no bond strength between the grout and the tendon. This was achieved by coating the tendon with a 0.1mm thick saponified wax. The majority of cone bolt trials tested were conducted on grout cube strengths of 30 to 50 MPa. From Rong this would equate to ranges 2:1 cylinder UCS of 23–26 MPa to 38–4 MPa dependent correction factors used.

Big Bell made a decision to undertake tests on eight bolts with their cone cut off. The bolts would have either an intact wax coating, or the wax coating physically removed. Embedment test lengths were 1.0 m and 1.5 m in approximately 35 MPa grout strength from 2:1 cylinder tests. The high peak load of wax coated bolt four (wc4) was found post test examination to have deterioration of the wax. The graph suggests that peak bond strength from the deterioration of the wax layer had a similar result to the non-chemical removal of the wax layer; however the residual strength of the wax deteriorated bolt was the same as the remaining waxed cone bolts. This was viewed positively.

The results were interpreted that the wax layer was important in the performance of the cone bolt, but a deterioration of the wax layer would not require the bolt to be scrapped.

An increase in bond strength was due to the removal of the wax layer. Variations in the performance of the waxed removed bolts was interpreted as possible variations along the diameter of the bolt, or slight bends in the bolt that are not apparent with a 'thick' wax layer.

The wax coated smooth bar had on average 25% less peak strength per meter of embedment than the wax removed bolts, and a residual bond strength less than 50% of the wax removed bolts.

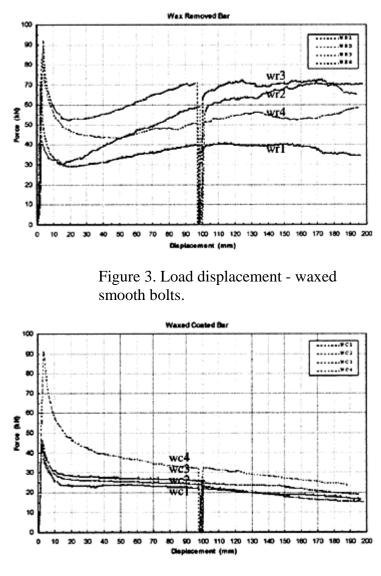


Figure 4. Load displacement - wax removed smooth bar.

Importantly there is not a nil bond strength between the shaft of the bolt and grout, but rather a low bond strength.

2.2 Bolt pull tests in 'soft' grout to determine tension times

Cone bolts were introduced as part of the development cycle in 2001. Each new sheet of mesh had cone bolts installed in the backs and the shoulders. Apart from the leading edge of the mesh which was against the development face. This edge would be meshed over and cone bolted with the next cut. The wall cone bolts were delayed for upto 20 m of face advance to reduce the potential of bolt damage from equipment loading out the fired round.

The use of the cone bolts in the development cycle required additives to rapidly set the grout. The product selected was Sikament HE200NN. Dosage rates of 1.25% and 2.5% with respect to weight of cement were assessed. Table 2 records the grout strength with time for laboratory prepared samples, with a water cement ratio of 0.35, general purpose cement and tap water. However mine water had total dissolved solids of 5500–6000 ppm.

Hours	1.25% HE200NN	2.5% HE200NN	
4	0.8	2.3	
6	4.1	20.8	
8	12.5	34.7	
672	36	51.7	

Table 2. Rapid set grout strength W/C ratio 0.35.

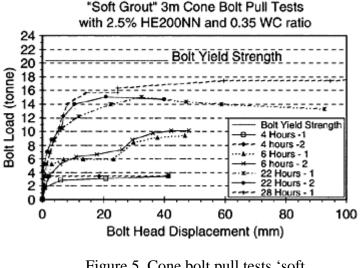


Figure 5. Cone bolt pull tests 'soft grout'.

Figure 5 shows pull testing of cone bolts in soft grout to determine the required set time prior to tensioning of the cone bolt.

The grout strength specifications from the manufacturer of conebolts 'Steeledale SCS' (trading on behalf of Grinaker—LTA) at the time stated: 'the conebolt is not highly sensitive to grout strength and will perform effectively in grouts with a uniaxial compressive strength ranging from 25 to 60 MPa, thereby the 22 mm unit will absorb up to 100 kJ without failing'.

The high grout strengths were not considered to be a significant issue from manufacturer supplied information. Big Bell test data, suggested that high strength grouts occasionally resulted in elevated pull out loads. However the low water cement ratio, low heat grout mixture worked in restraining rock burst damage in 1999 and 2000.

The installation procedures in late 2001, did not specify either low heat or general purpose cement, but set a water cement ratio of 0.35, a minimum set time of 6 hours for the grout prior to tensioning, required non-lubricated threads to be sprayed with a lubricant (WD-40) prior to tensioning, and only using the rattle gun on setting 2 and not stalling the gun during tensioning. Torque testing of non-lubricated threads after tensioning with the rattle gun recorded installed tensions of 50–65 kN.

Quality assurance 14 day UCS testing on 2:1 cylinders of contractor mixes in 2001 showed a variation of 42–72 MPa for good samples with an average of 52 MPa.

3 MODIFICATION IN INSTALLATION TO IMPROVE BOLT PERFORMANCE

The first modification was a reduction in the percentage of HE200NN utilised from 2.5% to 1.25% when using general purpose cement. This was due to problems of grout hardening in the bowl, increasing the difficulty of mixing the grout and then cleaning the bowl after grouting. The set time prior to tensioning was altered from 6 hours to 8 hours without any observation of the cone bolts 'spinning' or 'pulling' in the grout.

The second modification was to formalise the use of low heat cement rather than general purpose cement. The primary reason was to reduce the peak grout strengths that were recorded from the quality assurance testing program. It was considered that strengths in the 40–45 MPa range would perform better than strengths in the high 50's to 70 MPa. Additional work time with the grout mix was also obtained when using low heat cement rather than general purpose cement, the additive rate of 1.25% HE200NN was maintained.

With standard thick mix grouts it is difficult to obtain lower strength mixes. However lower strength mixes were considered to be advantageous as yielding of the cone bolt would primarily be the displacement of the cone through the grout rather than a combination of the cone displacement and steel yielding.

3.1 Understanding bolt breakage

The third modification resulted from a better understanding of the total ground response that the bolts were exposed to. It followed the observation of broken cone bolts in development headings with no visually significant ground deformation. The initial use of cone bolts at Big Bell occurred in competent footwall amphibolites that had already deformed during the development cycle, and prior to the onset of induced strain from the caving. Significant load transfer from shearing or dilating ground to the cone bolt did not occur prior to a seismic event. This gave the impression that the bolt was relatively insensitive to installation practices.

A footwall amphibolite extensometer recorded 2 mm of deformation over the length of a cone bolt prior to 20 mm of deformation occurring from relaxation with the passing cave front on the level. This can be compared to oredrive extensometer sites that typically recorded 60 mm to 80 mm of deformation associated with cave induced stress.

Typical examples of broken cone bolts are shown in Figure 6 to Figure 8. The bolts exhibited 'torque style failure' assisted by stress corrosion, from Collins 2002a and Collins 2002b.

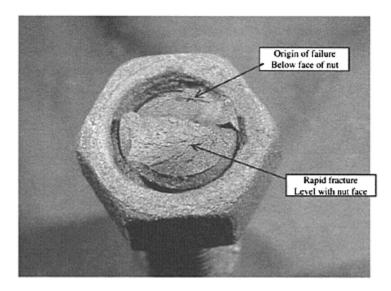


Figure 6. Close up of failed cone bolt.

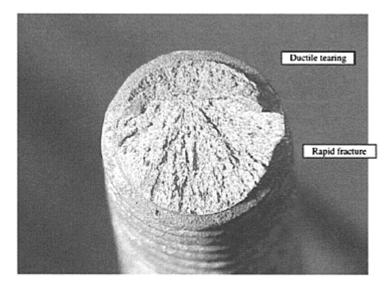


Figure 7. Cone bolt failed by bending.

Figure 6 displays the evidence of degradation during service. The failure has the appearance of high levels of cyclic loading, with the lower crack surface (upper side of the Figure) being present for some time. Corrosion was present in the crack tip and a corrosion fatigue mechanism appeared to be active. All bolts showed indications of pre-existing tearing possibly caused by the installation procedure.

The failure was attributed to 'tearing' of the steel during the installation process with stress corrosion cracking at the root of the tear. The tearing appears to have some cyclic loading component that could have been from the blasting of the development heading or impact loading with a stalled impact gun during the tensioning process.

3.1.1 Stress corrosion

The stepped failure path at the origin of the failure was some what unusual and advice was taken on the bolt failures, Collins 2002a, and Collins 2002b. The stepped failure was considered to be progressive in nature and due to stress corrosion cracking.

The nature of the steel supplied by the South African suppliers, meets Australian standards for steel 1035 and X103 8, however it does have a high level of non metallic inclusions stringers for modern steels that probably resulted from the sulphur level. The inclusions are elongated and in the direction of rolling of the steel. The stepped failure was similar to that observed in the occurrence of hydrogen induced cracking (HIC).

1. The factors that contribute to HIC are corrosion in acidic or oxygen free conditions. In the absence of oxygen reduction as the cathodic reaction, hydrogen is formed. This can diffuse into the steel, and combines in the inclusions to form molecular hydrogen which exerts pressure.

- 2. High tensile stress level due to the applied or residual stresses. Transverse tensile stresses caused by over torquing of the nut, or torquing against a high friction resistant thread, would balance out the high compressive stresses in the root of the rolled threaded. This allow the described progressive failure through the higher strength roll thread section followed by rapid failure once the crack reaches the lower strength core.
- 3. High inclusion content, especially if elongated in the direction of the rolling. These were found in all bolt samples that were examined.

The most important factors in reducing the type of failures observed would be through:

- 1. Reviewing the tensioning methods particularly with the impact air gun to ensure that excessive pre-tensioning is not occurring.
- 2. Use of cleaner steel, modern steels can typically have sulphur levels lower than 0.01%. Shape control can be practice to avoid the formation of long elongated inclusions.
- 3. Prevent or reduce corrosion by the application of a suitable corrosion preventative and lubricant on the threads.

3.1.2 Load applied by rattle guns

The impact rattle guns that are used underground typically use a linch drive. These guns can have three or four settings applying 680 Nm to 1630 Nm. However, if the gun is taken to stall on the maximum setting 2030Nm of torque can theoretically be applied to the nut.

This was a very large loading capacity when compare to what was required to torque the nut. Work by Thompson and Windsor 1999, shows how the thread friction must be kept low. With a minimum thread friction coefficient of 0.10, the axial stress comprises only 65% of the effective stress (35% in torsion). If the threads are not lubricated and the thread friction coefficient is about 0.25 then the axial stress only comprises about 40% of the effective stress (60% in torsion).

It can be estimated that the heavy oiled cone bolt thread when grit is embedded within the M24 thread would have a friction coefficient of 0.35 to 0.4. From the charts provided by Thompson and Windsor, 1999, to have resultant 50 kN tensile load to the bolt, only 600 Nm of torque was required to be applied, i.e. rattle gun on the lowest setting. However if 1000 Nm of torque is applied to the thread, there will be a reduction in capacity of approximately 330 MPa. The rolled threads have root strength of approximately 920 MPa, from Collins 2002a. This could allow a progressive crack growth through the work hardened rolled threads, followed by rapid failure through the core.

3.1.3 Bolt tearing from installation practice

The tearing was attributed to the following process. The cone bolts that arrived from South Africa, in groups of three in a plastic bag covering the threads, however the threads were unlubricated. Tensioning difficulties were encountered particularly after long storage of the bolts in which the bag covering the threads broke down. The operators were required to clean the threads with a wire brush and then spraying them with CRC lubricant, to enable tensioning. It was requested for the supplier to oil the threads. The bolt had a fine M24 thread and a thick oil was used, the bolts were still bagged in groups of threes. Dirt and grit became embedded into the thickly oiled threads is the bag was removed or broke down, effectively increasing the friction against tensioning the bolt rather than decreasing the friction.

Once realised, a second request was made of the manufacture that the bolts be:

- lightly oiled/lubricated,
- individually plastic wrapped rather then wrapped in groups,
- and that a courser thread be utilised that would not generate as high a torque should the bolt be over tensioned. There would also be less occurrence of material embedding into the thread.

The applied torque was reduced with a change in rattle gun setting to its lowest level, and re-instruction of the operators not to stall the gun during tensioning.

The bolt in Figure 7 was exposed to bending stresses prior to failure, these may have been the result of poor installation practices or from the exposed thread of the bolt being bumped by mobile equipment initiating the ductile tearing.

Figure 8 provides a clear example of the stepped failure process starting from the origin of the thread, with slow progress through the higher strength work hardened threads, with the assistance of stress corrosion, followed by rapid failure once the diameter is sufficiently reduced and the general bar strength is reached.

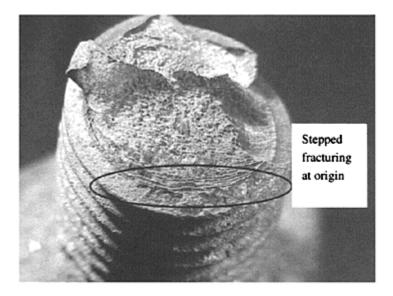


Figure 8. Stepped fracture at origin of bolt failure.

3.1.4 Rubber backing plates to apply uniform load dome plates

A rubber plate was introduced between the mesh and the twin dome bearing plates, rather than a standard w-plate. The plates were punched from 20 mm thick second hand conveyor belt that had a nylon ply core rather than steel for ease of cutting. They were approximately 250 mm diameter with a 50 mm diameter central hole.

The rubber plates greatly assisted in obtaining a uniform pressure applied to the dome plates from the ground movement. This was believed to be important because it increased the capacity of the surface plates and the cone bolt, as shown in Figure 9. Uniform pressure allows the double dome plates to flatten out and then dish. Point or localised loading of a dome plate, increases the probability of the plates being pushed over the washer and nut and/or introducing higher shear stresses onto the bolt and failing the bolt near the washer. There were limited instances of thread failure following the introduction of rubber backing plates rather than a standard w-plate.

The rubber plates also act as a dampener for the shock load onto the dome plates from seismic events and ground movement.

Ground pressure is shown prior to the flattening of the dome plates, with deformation of the rubber plate. In Figure 9 the two heavily domed plates were installed with the development of the drive, and the less domed plates were added latter to assist slowing structurally controlled wall deformation.

When the cone bolts are installed correctly it can be shown that they have a high load capacity. The requirement for two dome plates from South Africa came from simple load testing that showed a capacity closer to 180 kN rather than the required 200 kN when loaded axially and less than 160 kN at an angle of 15 degrees. This required that the plates be used as

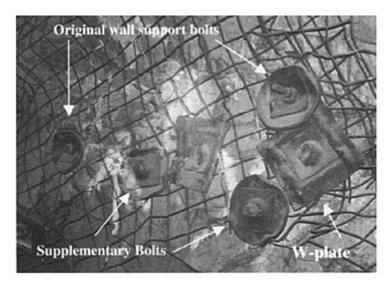


Figure 9. Heavy dome plate loading.

a pair, as the ideal yielding range of the cone should occur at 150 kN to 170 kN. As stated the integration of the support and reinforcing elements in seismic support systems is critical and extra capacity in the plates could be justified.

It was cheaper to obtain a pair of plates from South Africa then a single high capacity plate from Australia. The quantity of dome plates required meant there were occasions when alternatives to the South African plates where required, Australian alternatives that were tested to have a rating in excess 220 kN but these also could heavily deform. The heavily deformed plates on the right of Figure 9 are Australian 220 kN plates. The rubber backing plate is an important factor in the observed dome plate deformations.

Using a rubber plate between the dome plates and the mesh/ground, reduces the friction component of the dome interacting with the steel/ground and decreases the stiffness effectively softening the surface fixture by increasing the deformation of the steel dome plate when used in combination with a rubber plate for the same load.

If testing a ground support plate by pulling it into a cylinder (by ISRM standards 100 mm) the use of a rubber plate between the bearing plate and cylinder effectively increases the internal diameter of the cylinder.

This means that it could be necessary to use two steel dome plates following the introduction of a rubber backing plate, even if one dome plate has the nominal capacity of 220 kN.

The South Africa dome plates had a low dome angle with a flat seat for the washer, and would have a softer response to ground loading than the Australian steep sided dome plates with high angled seat for the washer.

Interestingly when galvanised dome plates had to be used apparently they approved to deform more readily than non-galvanised dome plates, this could

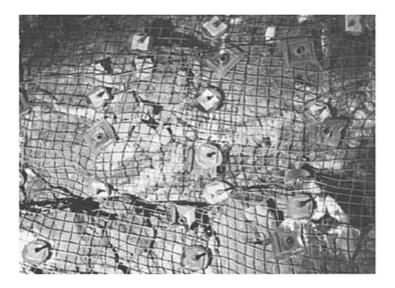


Figure 10. Deforming galvanised dome plates on a wall.

have been a function of the galvanisation process heating the plates.

Figure 10 shows a length of drive on the 560 level where galvanised plates where used on the wall, the dishing of the twin dome plates is apparent plus the deforming rubber plate.

Another item for consideration was that the cone bolts must not be 'freely sliding' through the grout in quasi-static conditions. Otherwise there would not be the significant plate deformation.

It could be that some of the high plate loading is associated to the use of General Purpose Grout rather then Low-Heat Grout, and shearing ground movement which locks the cone, but the wax coating and hence low bond strength on the shaft of the bolt allowed the surface rock of the excavation to push against the surface hardware.

4 DEFORMATION CONTROL 2002–2003

A quality assurance testing program of 174 cone bolts was tested in 2002–2003. Of the 174 bolts, 8% (14) failed. Failure defined as a residual pull out load of less than 100 kN. Of the failures:

- two of the bolts had peak loads above the 100 kN criteria but settled at a lower tonnage,
- one broke through the threaded length due to the reasons explained in the previous section.
- five of the bolts failures where identified as potentially poorly grouted prior to plating, and were specifically tested.

On the upper levels where the section of poorly grouted bolts were identified a length of development had their bolts replaced. The number of failures per level decreased as the mine got deeper showing an improvement in installation quality with increased operator experience. The lowest level had no failures.

The distribution of results from the 133 passing bolts taken to yield is shown in Figure 11. Another 27 bolts were taken to 120 kN and the test was discontinued.

Figure 12, details stress induced damage on the shoulder of an oredrive. The circled bolts are the original cone bolts and Splitsets. The additional cone bolts were added latter. The amount of deformation of the plates would be related to the ground moving but could also be because the broken ground behind the plate effectively allows a larger hole for the plate to be pulled against. In this case the Splitsets generally survived. But this was not always the case as shown in Figure 13, where two wall Splitsets have failed, with only low apparent load observed on the cone bolt.

The level of ground support and reinforcement installed in the strike drives initially seems excessive to the unfamiliar observer because of the apparent good ground conditions encountered prior to the effects of caving stresses.

Figure 14 is a view along an oredrive showing the general conditions and installed ground support. The oredrive is a nominal 4.5×4.5 with arched shoulders.

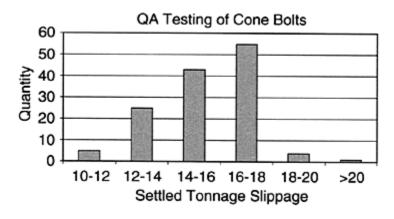


Figure 11. Cone bolt QA program.

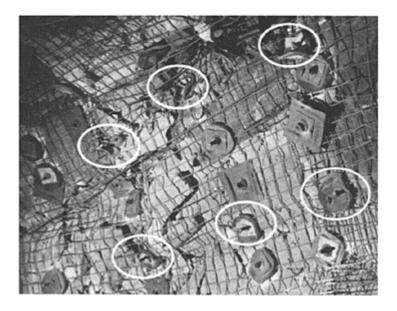


Figure 12. Stress induced shoulder damage.

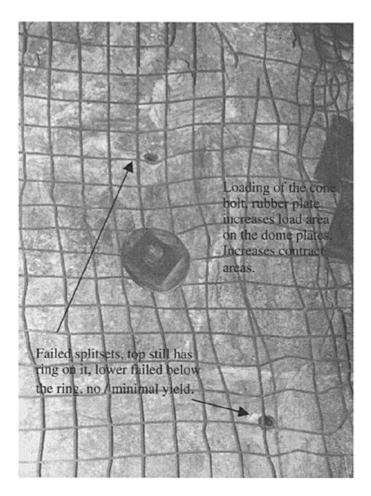


Figure 13. Failed wall Splitset from ground loading.

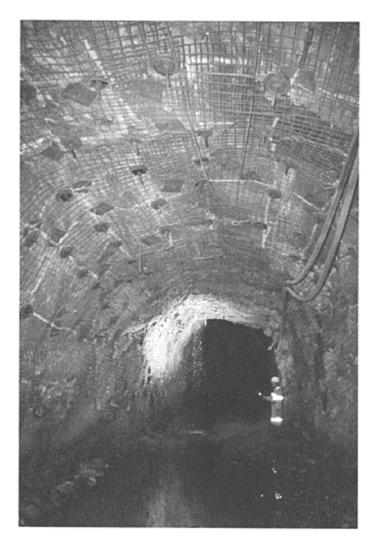


Figure 14. Good drive conditions before the effect of stress.

4.1 Integrity of wall support in production headings

The use of teleremote 2900 loaders in the relatively small drive size presented a problem for the survivability of the mesh and bolts, particularly those located at the end of turning curve for the loader as shown in Figure 15. This problem was address where fibrecrete was used the introduction of fibrecrete (with plastic fibres) around selected pillars corners

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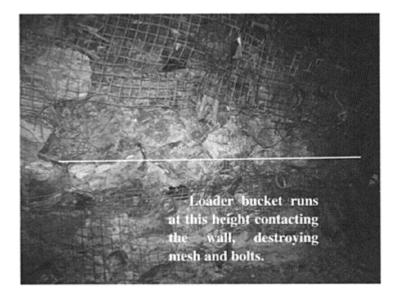


Figure 15. Loader damage to oredrive mesh.

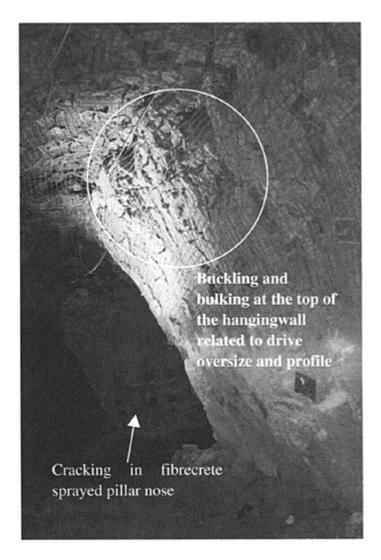


Figure 16. Fibrecrete lower walls of oredrive.

and at the toe of the walls for controlling wall breakout and unravelling from high stress on the 560 and 585 levels. Figure 16 and Figure 17 show examples of fibrecrete supporting of the walls and reduction of equipment damage to the support system. The author was away from site when the fibrecrete was done, and the selection process had not been fully documented.

Fibrecrete was sprayed onto existing mesh and Splitsets to interlock with the mesh, or specifically added mesh, and then bolted through with cone bolts or Splitsets (when the holes would not stay open for the cone bolts to be installed). This proved effective in controlling wall damage from loaders, stopping loader-induced rehabilitation of the drives where fibrecrete was used.

Should a mining operation determine that the walls must be supported, it would be worthwhile to undertake a cost benefit analysis of the cost of undertaking rehabilitation on a regular period at loader turn points as compared to spraying fibrecrete at the turning areas, and the line that the bogger bucket lip forms along the wall. Should this be undertaken, then spraying of the pillar nose would also be beneficial because pillars have an extra degree of freedom, particularly in the presence of shallow dipping structure.

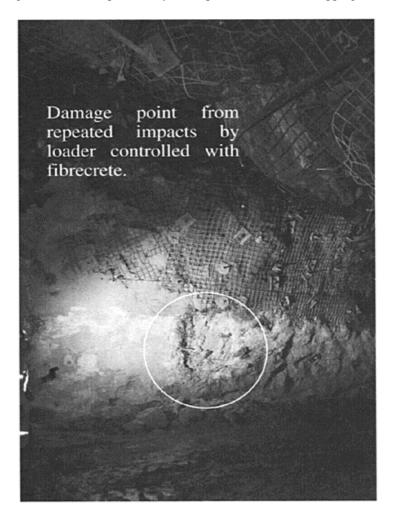


Figure 17. Fibrecrete controlling equipment damage.

5 CONCLUSIONS

In high deformation and seismic environments mine operators should expect difficulty in achieving 100% survivability of all reinforcement and support elements. Some redundancy in the support system is thus required.

The support system should fully integrate the reinforcing and support elements. Big Bell achieved this with cone bolts (approximate 1 m by 1 m pattern), M71 weld mesh (mesh overlap and join location), rubber plate (between mesh and surface plates) and twin dome plates.

An understanding and assessment of the ground response to stress with seismic and non-seismic deformation is important in understanding how the support system will perform.

Undertaking material failure investigations of reinforcing elements can be very useful in determining changes to installation practices, or how the element is required to be supplied to the mine site.

ACKNOWLEDGEMENT

I would like to thank Harmony Gold for the opportunity to publish the results obtained from field application of the cone bolts, and the learning process that the mine site went through in ensuring the ground control system functioned correctly.

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Performance assessment of tendon support systems submitted to dynamic loading

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ABSTRACT: Deep mines and underground operations with a high extraction ratio are sensitive to ground stresses. Increasing stresses in a mining area can cause the failure of rock masses around openings and convergence of rock towards the exposed surfaces of excavations. Under a combination of circumstances such as the presence of strong and brittle rock and high stresses, it is possible for the rock mass to fail violently thus causing a rockburst.

Reinforcement and tunnel support means such as the use of rock bolts, reaction plates and mesh are well suited for gravity driven situations when the support systems are subjected to static and quasi-static loading. The performance of the systems can be at any time verified, through pull out testing of the tendon support elements and the meshing units for example. Bolting patterns and support system design can be inspired from a number of published references to meet specific performance requirements.

The design of support strategies for dynamic loading has been for now less obvious. Published guidelines suggest adopting a tendon burden as well as an arbitrary ejection velocity. These factors can be used to calculate the energy absorption capacity requirements of a support system so that it can be compared to the work energy during a pull out test in quasi-static loading mode.

In this article a different approach is proposed based on quasi-static and impact pullout testing data assembled during the MCB (Modified Cone Bolt) prototype tendon support validation. Results and observations from the testing phase were used to derive a displacement evaluation method for tendons submitted to impact loading. The proposed displacement evaluation method could provide a means of calculating the maximum capacity range of tendons for rockburst support design.

1 TUNNELS IN HIGHLY STRESSED GROUND

1.1 Background

The tendon energy absorption testing methodology presented in this paper was developed concurrently with Noranda Inc.'s Technology Centre (NTC) research project R2-9684. The project related to the design of support systems able to sustain rapid loading conditions. In 2002, a patent application (US 6,390,735 B1) has been granted for Noranda Inc. regarding a new bolt called the Modified Cone Bolt (MCB). This new tendon can yield in its polyester resin grout matrix. The bolt's mechanical response in rapid loading has been evaluated using an impact test rig located inside its NTC premises. Impact testing on different bolt types called for an energy absorption calculation method that could describe the tendon's impact behaviour through time for two reasons. The first is due to limitations on the test rig's maximum drop weight load and height. Certain types of support require more than one impact to break the tendon. Thus drop heights and impact velocities were not constant from one test to another. The second reason pertains to the purpose of the impact testing. The characteristic response of each tendon had to be summarised in such a fashion that it could be used for rockburst load simulation purposes. This paper focuses on the selected method for testing and assessing the response of different systems to impact loading, by proposing a methodology for the verification of the MCB capabilities (Gaudreau 2004).

Tendon support is most often used with other support elements, such as wire mesh and straps, as a system for tunnel support in mines. Tendon support consists of a stiff rod of a given geometry, length and diameter installed in a borehole. Different tendon support systems are available, for example cable bolts, rock bolts, rebars and Split Sets (e.g. Hoek et al. 1995). Each type of tendon will react differently when subjected to slow or rapid loading.

Stiffer support systems can provide immediate resistance to the deformation of openings. If the deformations become too large or if the drift sustained damage, the tunnel is rehabilitated, usually by removing loose parts and installing new rock support devices. Certain support systems are better suited to carry massive deformation through time, and can avoid the rehabilitation process. By definition, a yielding tendon support system has enhanced load-time distribution properties when subjected to large displacements, while providing resistance to the movement. An example is given for clarity. A massive impact load is suffered by a supporting structure at the periphery of a tunnel, due to the violent failure of a wall. If the wall support system is stiff, it will carry the full load in a very short amount of time. Hence, the steel tendons will elongate elastically then break, leaving a large amount of energy available that may be imparted to mobile rock particles. The smaller ejected masses will likely be ejected at high velocity, the heavier ones will be found near the toe of the wall. If this wall was supported using yielding support, the impact load received by the reaction plates through strapping and screening materials will be transferred to the tendon unit. If the tendon unit can move and follow the wall displacement with some resistance, the load will be transferred to the tendon, until the wall comes to a full stop. It is near the end of the deceleration that the tendon will sustain the most damage if the dynamic load is too large. But during the controlled movement of the wall, a quantity of work energy will have been absorbed,

which could decrease the damage to the opening. The load absorbed inside the tendon is a complex mechanism dependent on the characteristics of the support system and of the bonding materials inside the borehole. One can speculate that the pressure of the reaction plate and straps on the wall in reaction to the tendon displacement inside the borehole can reduce the size of the failure zone since rock is stronger when confined. The extra confinement given by the movement resistance of the tendons could change the size of the failure zone during the impact.

All support systems available commercially have limitations, and the choice of the proper one depends on factors such as the in-situ stresses, the expected rate and amount of deformation of the drift walls, the nature and quality of the rock mass, as well as the tunnel function and utilization time.

Cook and Ortlepp (1968) suggested the use of yielding support in the deep gold mines of South Africa. The concept was further developed by Jager et al. (1990), who introduced the South African Cone Bolt (Jager 1992), a groutable tendon equipped with a cone anchor. Preliminary impact testing of resin and cement grouted Cone Bolts was conducted in May of 1998 at the Noranda Inc. Technology Centre. Testing results and other industry results suggested that the Cone Bolt was not reliable when installed with cartridged polyester resin, but seemed effective for use with cement grout. This was mainly due to the bolt's inability to mix the cartridged resin in a reproducible fashion.

1.2 Scope of work

This paper presents experimental procedures and results pertaining to the evaluation of tendon support performance in impact loading. More specifically, the experimental results and procedures will be descriptive of the validation tests for the Modified Cone Bolt (MCB) developed at Noranda Inc. Technology Centre (NTC).

Section 2 contains a literature review on seismicity in mines, load-deformation-time behavior of tendon support, and testing methods thereafter. Section 3 contains experimental procedures used to test MCB prototype tendons at various loading rates, as well as testing results and analysis. Section 4 elaborates on a proposed displacement evaluation method for tendons submitted to dynamic loading based on observations from impact testing of MCB tendons.

2 LOADING MECHANISMS AND REQUIRED PERFORMANCE OF TENDON SUPPORT

2.1 Rockbursts and seismicity in mines

In Eastern Canada rockbursts are often called "bumps". A 1920 definition of a bump is "a sudden breaking sometimes accompanied by a setting or upheaval of the strata in the mine, accompanied by a loud report. (...) often interpreted as a sudden squeeze, or buckling of the floor or walls of the mine passage-ways. It has its origin in the shocks accompanying earth movements" (Fay 1947).

Rockbursts are violent failures of rock that result in damage to excavations (e.g. Cook 1965). Only those events that cause damage in accessible areas of the mine are called

rockbursts (Gibowicz 1993). Out of thousands of bumps recorded in Canadian mine seismic networks, only a few can be considered as rockbursts.

A seismic event is a broader term referring to all occurrences that are associated with the release of kinetic energy, with the exclusion of blasting. Salamon (1974) suggested that a rockburst could be described as a seismic event that adversely affects the operation.

2.2 Rockburst source mechanisms, size and classiflcation

Here, the author suggests a rockburst mechanism classification in Table 1. This classification was derived from Ortlepp (1992) to better reflect the relative calibration of Noranda Inc.'s mine seismic systems. The classification is illustrated in Figure 1. Note that this classification is somewhat arbitrary. Its intent is only to illustrate the relative magnitude of what could be "felt" by someone near the affected area, in terms of a fairly well known moment magnitude scale, namely the Richter scale.

As illustrated in the classification, different rockburst mechanisms have different associated "burden" on the tendon support system used in the tunnel. For example, the mechanism "face crush", is probably the one which could have the highest individual tendon burden and impact velocity on the support system. The size of the volume of rock that violently fails and its proximity to an open face can cause a lot of damage. On the other hand, the mechanism "fault-slip", although corresponding to the highest probable moment magnitude, is the one less likely to induce a large burden or impact velocity on the support

Type of failure	Type of failure Postulated mechanism at the source of the seismic event	
Stress-induced fracture	Energy dissipation in the rock mass by creation of new fractures	-3.0 to -1.0
Strain bursting	Superficial spalling of tunnel surface and violent ejection of rock particles.	-2.0 to 0.0
Buckling	Bending of rock slabs inwards the tunnel due to the pressure on both ends	0.0 to 1.5
Face crush/Pillar burst	Violent and deep expulsion of rock from a tunnel surface or multifaceted structure (pillar, rib, "skin" pillar, remnant)	1.0 to 2.5
Shear rupture	Violent propagation of a shear surface in a solid or healed area	1.0 to 3.0
Fault-slip	Violent slip on a pre existing shear surface.	2.0 to 3.0

Table 1. Proposed classification for rockburst source mechanisms and associated damage.

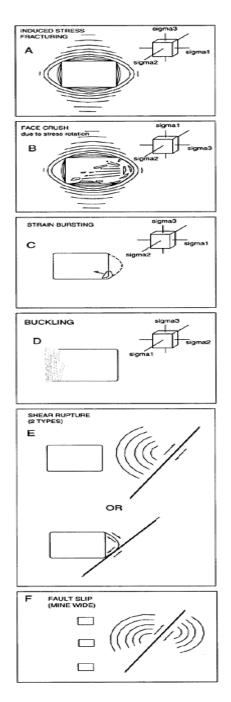


Figure 1. Tunnel damage classification (Gaudreau, 2004).

system. Its incidence on tunnel stability is most likely to be due to shake damage or spalling of loose and poorly supported material.

2.3 Performance requirements of yielding support

Tunnels driven into highly stressed ground typically suffer from stress-induced damage (e.g. Wagner 1984, Mühlhaus 1990, Ortlepp 1992a, Dyskin & Germanovich 1993, Langille et al. 1995, Maxwell & Young 1997). Stress-induced damage can form from either creation of new fractures or reactivation of existing fractures in the rock mass. This phenomenon is depicted in Figure 1a, showing an exaggerated profile of the induced fracture lines relative to the influence of the principal stress direction.

The extent of the induced fracture zone can affect the amount of "dead load" in the back of the tunnel. Static loading performance of tunnel support in highly stressed ground must reflect this aspect. Further to this, the tunnel support must be able to tolerate relative convergence of tunnel walls, or swelling and squeezing rock conditions (Stillborg 1986). Sidewall dilations in excess of 500 mm have been recorded under both static and dynamic conditions. However, when such large movements occur, the dimensions of conventional tunnels are reduced to such an extent that the basic functions of these tunnels cannot be maintained. Wojno et al. (1987) recommended a static tendon yield force greater than 100 kN to control wall movement. Moreover, these same authors suggested that the distribution of the dilation within the fractured zone is an important factor to the design of support tendons for yielding ground. They observed average values of dilation in a highly stressed tunnel to be such that 46% occurred within 2 m of the tunnel wall, 16% between 2–3 m and 38% at depths greater than 3 m from the tunnel sidewall. They thus recommended that the yielding range due to sidewall/hangingwall movement within the supported rock thickness for non-rockburst conditions be:

- 230 mm for a tendon length of less than 2 m,
- 310 mm for a tendon length of 2 m to 3 m,
- 500 mm for a tendon longer than 3 m.

Langille et al. (1995) have set the following criteria for the selection of a one-pass support system (i.e. that is installed in one cycle) for use in a high stress mining environment at Creighton Mine in Sudbury:

- immediate support of loose rock blocks for protection of men and equipment at the face,
- yieldability of at least 50 mm in the short term,
- long term rigid reinforcement of the broken rock mass,
- corrosion resistance to heat, humidity, fumes, smoke and percolating mine water.

The quantity of material that could potentially be statically contained by a rockburst support package after a dynamic event will depend on the bolting pattern at the periphery of the opening, or more specifically the tendon burden. Tendon loads can easily reach the order of 10 tons. Kaiser et al. (1992) have proposed a Rock Damage Scale for which levels of displaced rock range from a few kilograms to amounts greater than 10,000 kg.

Further to quasi-static considerations pertaining to "dead load" on tendon support, one must consider dynamic effects if a tunnel is to be subjected to high stresses. Dynamic

loading implies physical forces producing motion. Stress changes and blast vibrations after mining to a new stope geometry can produce dynamic loading of a nearby tunnel. The stress regime at its periphery can fluctuate rapidly.

Wojno et al. (1987) have set these guidelines for the capacity of tendons in dynamic loading:

- The amount of work to be done during yielding of the tendon must be greater than 25 kJ.
- The mean dynamic yield force must be in excess of 50 kN.
- The maximum dynamic yielding range must not exceed 500 mm of displacement.
- The dynamic strength of the tendon should exceed the static yield load of the tendon by at least 25 percent.

Gaudreau (2000) set the design criteria for the components of a yielding support system for Noranda Inc. at:

- peak reaction load of less than 11.3 tons and greater than 6.8 tons for an impact energy of 15 kJ;
- tendon system plasticity limit at load greater than 6.8 tons;
- pull-out displacement greater than 150 mm (at maximum static capacity and at impact loading of 15 kJ);
- ability to install in cement or polyester resin grout using mechanized or nonmechanized means of installation;
- support must not creep if load below plasticity limit after initial movement of the anchor;
- better corrosion resistance than resin-rebar installation;
- support can be pre-tensioned at installation;
- support to be installed in a 38 mm hole with a 17 mm smooth bar of grade C1060.

These criteria were set to match energy absorption and wall control requirements from damage observed underground at Brunswick Mine and from other operational restrictions including compatibility with the mine's machinery and rapidity of installation. The choice of the smooth bar was made based on two assumptions:

- it is preferable for the full rock load transfer in the steel rod from the reaction plate to the bolt's inner end,
- there is a possibility for creating an "active" support effect, i.e., additional clamping charge transferred to the rock mass during the event of a strain burst from a tunnel face.

The latter assumption could be verified provided the demonstration that the volume of rock subjected to failure decreases with the instantaneous clamping provided by the bolt during the burst.

The use of energetic absorption requirements in the formulation of yielding support performance requirements can be estimated by different means. For example, one can use case analysis of rockbursts, or published data pertaining to average ejection velocity (Yu 1980, Wagner 1984, Stillborg 1986, Wojno et al. 1987, Ortlepp 1993, Kaiser et al. 1996). A different, and perhaps more pro-active method could be to evaluate the ERP (for Evaluation of Rockburst Potential) for the mining area and calculate (Simon et al. 1998, Simon 1999) the quantity of excess energy available after rupture using stiffness comparisons. Once the quantity of excess energy is approximated, and given a tendon support burden, one can calculate the average possible impact velocity on the support system. The importance of the proper selection of the impact velocity will be demonstrated in section 4.

2.4 Performance of yielding support

Commercially available tendons have been tested for their plausible reaction to impact loading using different methods. Tendons are often pull-tested and the load-displacement curve can be utilised to calculate the work spent during the test. Tannant & Buss (1994), Langille et al. (1995), Kaiser et al. (1996) and Hoek (2000) published pull strength parameters for tendon support and mesh. The different tendons are classified by their ability to yield. Split Sets, Swellex and Cone Bolts were classified within the best yielding tendons available on the Canadian market, thus would be better suited for use in rockburst-prone areas to prevent damage.

Tendons can also be directly tested in rapid loading. Most tendon rapid loading testing methods depicted in the literature can be classified under explosive loading or impact loading categories.

Ortlepp (1992) discussed the blast testing approach where a tunnel half-section is bolted with conventional end-anchored bolts and the other half with yielding endanchored bolts. These trials were conducted by Ortlepp in 1969. The test was used to demonstrate the relative performance of both support systems under dynamic loading. Another widely published experiment concerns the explosive testing of the COMRO Cone Bolt, developed and manufactured by Strata Control Systems, and now manufactured by Steeldale in South Africa (see Ortlepp 1994, Stacey & Ortlepp 1994, 1999, 2000, Ortlepp & Stacey 1998). In this experiment, six different sets of support tendons, namely two distinct sets of 16 mm Cone Bolts, and one set of each 16 mm rebars, 16 mm smooth bars, 25 mm rebars and 22 mm Cone Bolts were installed in a quarry. Each set was installed and grouted into bedrock through an independent concrete slab. Explosive charges were then inserted in horizontal cavities consisting of rows of PVC pipes laid on the bedrock before the casting of concrete. Each slab was tested separately and monitored using a high-speed camera and a velocity transducer consisting of a velocity of detonation electronic timer. The objective of the experiment was mainly to compare the performance of a stiff support system to a yielding one. The explosive jolt was not the same from one test to another, making it difficult to estimate the energy absorption of each set. Gases from explosive charges did not burst out from the fractured ground in exactly the same way every time, thus inducing a different energy at each test.

Tannant et al. (1992) used a different approach when a test drift, located near a large stope, was instrumented in the expectation of a large blast susceptible of triggering seismic activity. Accelerometers were set up directly on Split Sets, mechanical bolts and Swellex bolts. The performance of tendon support under these events was thus measured. The peak particle velocity measured was in the range of millimetres per second, whereas ejection velocities during a rockburst are in the order of 1 to 10 meters per second. Although this technique measures the impact of real seismic events on tendon support, it

carries a disadvantage, the uncertainty of expectation. One must wait until a natural seismic event hits a particular instrumented area to get results.

Ansell (1999) and Tannant et al. (1994) have worked on an approach were the dynamic damage is simulated in a tunnel by using a blasthole drilled at a small angle to the axis of the haulage. The maximum ejection velocities were no greater than 2 meters per second, but were successful in creating damage to the haulage. The tests showed that loading by explosives close to bolts causes cracking of the surrounding rock and thereby inadequate loading of the bolts. Furthermore, it would be preferable to test support packages at velocities higher than 2 m/s for rockburst support design.

Special impact test rigs have been used to test tendons at impact velocities and loads that are comparable to these of yielding support system requirements. Anders (1999) grouted yielding tendon support inside a large cylindrical concrete mass. The mass, attached to an horizontal H-beam, was dropped from the ceiling of a two story high pilot plant, to a receiving structure were it was suddenly stopped. The set-up was used for fully grouted bolts and for ungrouted steel bars coupled to steel weights.

Stacey & Ortlepp (1999) used a swing-beam mechanism and a large mass to provoke the separation of a test tube where the tested tendon is installed. Maloney & Kaiser (1996) have designed the test rig that has been installed at Brunswick mine in 1997 and was later modified by Noranda Inc. The test rig was never used at the mine site due to operational difficulties.

Tendons have also been tested in shear and dynamic shear. Tendons can be loaded not only in pure traction, but also in shear or by a combination of both loading modes. This can occur during the course of induced stress fracturing (Figure 1a) and dislocation of fracture surfaces thereafter. The shear movement can be significant if combined with stress changes due to mining of excavations in proximity. Haile et al. (1995) have studied the phenomenon using two types of shear inducing apparatus to perform shear tests on tendon support. The tests were performed under both static and dynamic loading. It appears from the results that the 16 mm Cone Bolt and the 16 mm smooth bar performed equally well under dynamic shear. The 16 mm Cone Bolt showed better shear resistance in static shear than the smooth bar. The authors have recommended the Cone Bolt over the smooth bar, rebar, Split Set and twist bar in applications where shear movements in the rock are predicted. They recommended further work before using larger diameter bars for shear loading but argued that it would not be necessary to increase the diameter size of the Cone Bolt for such a use. The strength of the Cone Bolt in shear loading mode lies in its ability to alter the loading mode. In essence, the Cone Bolt installed in a dislocating wall would be loaded in a combined shear and traction mode.

Gillerstedt (1999) experimented on the ability of the 22 mm Cone Bolt to perform in mixed loading mode. The bolts were installed through two concrete blocks and loaded in shear. The resulting axial load on the tendon was measured using a load cell mounted under the bolt's reaction plate. Electronic displacement transducers were used to measure the cone displacement in the grouted bore hole and the crack aperture. Wire potentiometers were used to monitor shear movement of the concrete block. The deformation rate was approximately 0.1 mm/s. Results indicate that the Cone Bolts did transfer some of the shear load into axial loading. One particular Cone Bolt sample broke in shear after 226 mm of shear displacement having locked in place in its encapsulating

matrix. The compressive strength of the cement grout did not seem to influence the Cone Bolt's behavior by comparing results.

3 EXPERIMENTAL PROCEDURES AND MAIN RESULTS

3.1 Quasi-static underground pull-testing of MCB tendon

Quasi-static pull testing is generally used to evaluate the in-situ reaction of tendon support. This consists in extracting pre-installed rock support from its installation site using a hydraulic ram while measuring the force given to the ram and the displacement of the tendon at the collar of the hole. A schematic of the pull testing assembly used for the prototype MCB tests is illustrated in Figure 2.

Pull testing was performed in April 2000 at Brunswick mine on MCB prototypes of 2.1 m length. Displacement was measured through a potentiometer and load using an electronic pressure transducer, as illustrated in Figure 2. The polyester resin used was Fosroc 35 mm diameter cartridges. A summary of quasi-static pull test results is illustrated in Figure 3.

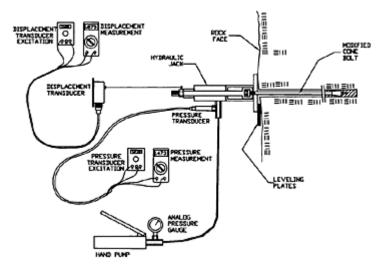


Figure 2. Diagram of pull test assembly.

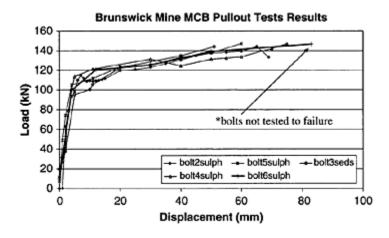


Figure 3. Summary of quasi-static pullout test results performed at Brunswick mine.

The testing results showed good agreement with the design criteria. The bolting system was neither too stiff nor too flexible.

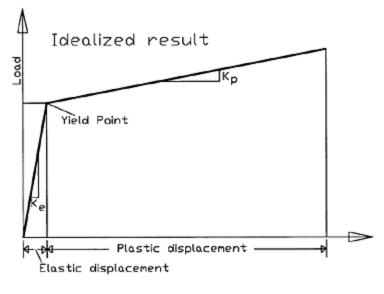
The instrumented pull test result consists of a load displacement characteristic curve representing the possible performance of tendons under the same relative displacement. Such a schematic characteristic curve is illustrated in Figure 4.

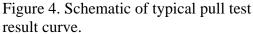
Typical rock tendon behavior under quasi-static loading show an elastic displacement phase and a plastic displacement phase. For the benefit of this study, the idealized tendon reaction will be said to have an elastic stiffness K_e and a plastic stiffness K_p . Some permanent displacement is said to occur when a tendon has been pulled past its elastic displacement range into its plastic deformation range. The plastic stiffness K_p as can be measured in a quasi-static pull tests will be used later to estimate the tendon support's reaction to impact loading. If a pull test is done for that purpose, it is important to pull the tendon long enough to gather a significant part of the plastic deformation behavior without necessarily carrying it to failure which can be hazardous.

3.2 Impact testing

The impact testing facility located at NTC used a drop weight to induce displacement of the tested tendon at a fixed initial impact velocity. For impact testing, the rig can be set for a drop weight of a maximum of 1000 kg over a fall distance of no more than 2 m. The potential energy is thus of a maximum of 20 kJ. Figure 5 illustrates the details of the impact test rig.

The drop weight is attached to a release system located on the top part of the facility. Once released, the annular shaped impact mass slides along the test tube until it collides with the reaction plate. The latter is supported by the tendon installed in a test tube of an internal diameter of 38 mm and of 9.5 mm wall

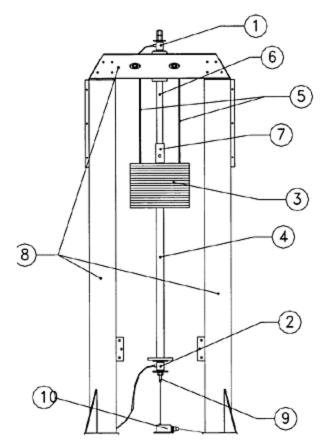




thickness. Each sample is prepared and installed as it would be underground using a stoper mounted horizontally on a track. A stoper is a hand-held mining drill that can be used to drill boreholes and install roof support.

All tests were instrumented using a load cell located on the top part of the machine, reading load on the test tube, a load cell underneath the reaction plate, reading load on the tendon, and a potentiometer attached on the tendon to read displacement of the bar. The steel mass is hoisted into position using a 5 ton capacity crane.

All 1.7 m long MCB prototype samples were spun into two Fosroc 915 mm length and 35 mm diameter resin cartridges. The first cartridge has a speed index of 30 (for fast), and is of type 35915M35. The second has a speed index of 240 (for slow), and is of type 35910LIF90. All samples were pushed through the



- 1-Load cell 1
- 2-Load cell 2
- 3-Drop weight
- 4-Sample holder
- 5-Holding rods
- 6-Spacer
- 7-Coupling
- 8-Frame
- 9-Tendon (in sample holder)
- 10-Displacement transducer

Figure 5. NTC Impact test rig schematic.

slow setting resin, then pushed and rotated through the fast setting resin, to simulate a jackleg/stoper installation underground. Load-displacement curves generated from the tests are based on the measurements made from the superposition of load curves from the top and bottom load cells. The surface area under the load-displacement curve representing the work energy dissipated is typically calculated from the bottom load cell when available.

Data was recorded using a LeCroy 9424E Quad 350 MHz oscilloscope. Three channels were used for the test, two for the load cells, and one for the displacement transducer. The top load cell was an RST model SGA-75–1.30 of 34 ton capacity. The bottom load cell was a Sensotec model TH/1591-01 of 90 ton capacity. Each cell was connected to a Vishay P-3500 strain indicator. The amplified signal was recorded using the oscilloscope. The displacement transducer was a UniMeasure P-20A potentiometer.

Although a large number of data points were collected in little time (40,000 data points in 0.2 seconds), the waveform recorded was noisy. The source of the noise is assumed to be due to environmental electrical and magnetic disturbances whose source was generated by machines and power distribution around the pilot plant at NTC. Voltage jolts often triggered the oscilloscope, and the amplitude and frequency of the ambient noise was erratic. A wavelet filter provided the ability of rejecting the white noise and was considered well suited for the transient nature of the recorded displacement and load waveforms. The removal of noise from noisy data to obtain the unknown signal is referred to as denoising. The wavelet shrinkage method (Wolfram Research, 1996) was used to suppress the white noise.

Figure 6 illustrates one impact test result on a MCB specimen. The figure illustrates processed filtered signals from ONE test, in load-displacement form. Displacements and loads have been combined on the graph from which work energy can be calculated as the area under the curve. The oscillations, mostly detectable on the top frame load cell (force1), may correspond to the natural frequency of the tendon-tube-impact machine system.

Specimens were typically tested numerous times, thus using numerous cycles of loading. Permanent damage to the steel was only detectable if the load measurement at the end of each loading run exceeded the elastic limit of the steel material.

Impact tests demonstrated that the MCB prototype could absorb an impact load of 1000 kg over 1.5 m drop height, resulting in an impact speed of 5.4 m/s. Multiple impacts were imparted on each specimen. Since the specimen's reaction plate was driven further down at each test, the impact velocity was changed at each loading cycle. Can one add the absorbed energy from each test to conclude on the energy absorption capacity of the MCB tendon? It appears that testing results are influenced by the impact speed and the size of the drop weight, but to which extent? Table 2 contains a brief summary of testing results on five different MCB samples. In section 4 a method is proposed to calculate the energy absorption capacity of tendon support systems submitted to dynamic loading as a function of tendon burden and impact velocity.

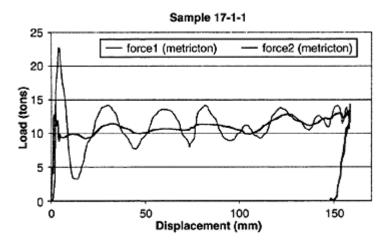


Figure 6. Impact test result on specimen 17-1-1.

Table 2. Tendons tested to failure and tendon load
measured during last cycle of loading.

Sample	Cycles to failure	Energy absorption by cycle (kJ)	Total additive energy (kJ)	Load at failure (tons)
17–3	4 at 1000 kg	16.4+17.5+18.3+7.4	59.6	Not available
17–11	4 at 1000 kg	17.3+17.0+12.7+3.2	50.3	19.6 thread failed
17–12	4 at 750 kg	11.0+12.0+12.3+10.6	45.9	23.7 nut slipped
17–13	4 at 750 kg (no failure)	11.6+12.1+12.9+14.2	50.8	Not applicable
17–14	5 at 750 kg	10.6+11.4+12.3+13.0+12.4	59.6	23.0 thread failed

It will be demonstrated using the methodology presented in section 4 that the choice of the drop mass and impact velocity can affect the total additive energy calculated for a tendon support system.

4 PROPOSED DISPLACEMENT EVALUATION METHOD FOR TENDONS SUBMITTED TO IMPACT LOADING

In this section, a displacement evaluation method is proposed for tendons submitted to impact loading, given a tendon burden, impact velocity, the tendon's yield point and its plastic stiffness under quasi-static loading. The method can be used to estimate the tensile load within a tendon submitted to axial impact loading. The proposed calculation method

could be used to interpret the maximum energy absorption capacity of the tendon for a single impact, and to evaluate the maximum load in the tendon for a given choice of ejection velocity and tendon burden.

The ejection velocity could be derived from energetic considerations equations, given a plausible violent rock mass failure of a given depth and expected energy. The burden of the tendon, or impact mass, can be estimated from the depth of failure expected in the rock mass and the bolting pattern applied. One could evaluate if the tendon can withstand the given impact, by verifying if the axial load in the bolt exceeds its ultimate tensile strength.

An adequate model for tendon displacement submitted to impact loading is required to analyze impact test results and could ultimately be used to predict the energy absorption capacity of support systems. Cyclic impact testing was performed on the MCB bolting system. Although it would have been interesting to set the impact testing for higher impact energy, so to break the tendon on first impact and directly evaluate its maximum energy absorption capacity, the available equipment could not be set for higher drop heights nor bigger drop weights. The relative effect of an increase of the impact velocity and/or the drop height in the testing protocol on the tendon's response must also be assessed. For the benefit of the MCB testing program, the velocity for all tests was higher than 5 m/s, and increasing at every cycle of loading for one sample since the tendon reaction plate was pushed further down with each blow. Drop loads were varied between 750 kg and 1000 kg. The impact tests on the MCB prototypes have provided valuable information useful for the selection of a calculation method for the displacement of tendon support submitted to impact loading.

The following phenomena were observed during the impact tests protocol:

- The impact load-displacement curves show an oscillatory behavior, possibly representing the exchange of load from the nut to the cone in a number of pulses through time. Ideally, the calculation method should reflect the oscillatory behavior.
- When the so-called maximum load reaches a value larger than the static yield point of the steel, the carbon coating on the steel bar flakes off, suggesting plastic deformation of the tendon. Ideally, the model should be applicable beyond the elastic range.
- There are no signs of plastic deformation on steel bars for which the initial impact peak load (see Figure 6) was larger than the static yield point of the steel.
- The bar can slide further into the test tube even at the last impact cycle on a sample,
- when the yield point of the steel is almost equal to the ultimate tensile strength.
- Discrepancies in yield load were observed at the onset of movement.

The discrepancies in yield load from one test to the next can be due to the adhesion of the tendon in the resin matrix. If the adhesion is strong, the load transfer from the reaction plate to the conical anchor is not completely achieved before the plastic deformation wave travels back to its origin. This would result in premature deformation near the collar of the test tube, and a higher load response than normally measured at the onset of movement. This can momentarily create uneven steel properties over the tendon length. Overall, the observed phenomena suggest that the tendon support system materials can be hardened due to plastic strain, and that the impact cycles have no apparent effect on Young's modulus or on the ultimate tensile strength of the steel.

The proposed calculation method for the displacement evaluation of tendon support submitted to impact loading is based on critically damped harmonic motion (Engel 1978, Derrick & Grossman 1987, Thomas and Finney 1992, Van Sint Jan 1994) and incorporates a so-called "friction factor" and a yield point offset. The critically damped harmonic motion model consists in a single spring and a dashpot attached to fixed points at each distant end and to a mass at the inner end. A spring and a dashpot in series constitutes a Maxwell model (Mase 1970; Gibowicz 1993). A Maxwell substance, or an elasticoviscous material, behaves differently under rapidly changing stress in contrast with slow loading. The MCB support system behavior displays this characteristic. The friction factor is a force referring to friction loss and heat dissipation during impact loading, it is not a yield point. Thus unlike a true Maxwell substance, the proposed model incorporates the combination of the so-called friction force of the tendon in the holding matrix and the action of the yield point when the tendon goes beyond the plastic range.

The proposed calculation method consists of two main steps. The first relies on a rheological model used to simulate the displacement of a tendon under impact loading. The second involves potential energy and work balanced with the friction factor imparted to the rheological model. The outcome is the approximation of the maximum tendon displacement under impact loading, or alternatively its maximum axial load for a given mass, impact velocity and characteristic plastic stiffness. The rheological model chosen to simulate the displacement of tendon support under

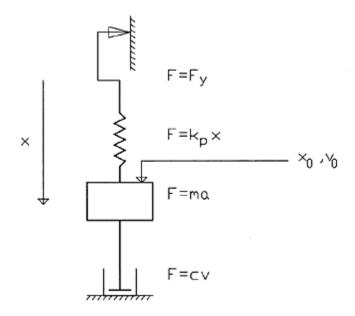


Figure 7. Rheological model.

impact loading consists of a mass attached to a slider, a spring, and a dashpot in series (Figure 7). The mass m is impacted at initial velocity and displacement v_0 and x_0 . The spring has a plastic stiffness of k_p to which is added a constant yield force of F_y when

displaced. The dashpot has a damping factor c, proportional to the plastic stiffness of the spring and to the size of the impact mass. Positive displacement x is downwards.

The overall constant yield force value Fy (N) consists of:

 $F_v = YieldLoad + F_f$

(1)

where YieldLoad is the yield load (N) of the tendon material and F_f is the friction factor (N) representing all sources of friction losses.

The model should then require only input parameters that are known a priori from quasi-static pull testing results and specifics, such as the impact mass and initial velocity, the tendon support system plastic stiffness, and the yield load of the tendon material.

Solving the equilibrium of this model (e.g. Figure 7) for forces, we have:

$$m\frac{d^2x}{dt^2} + c\frac{dx}{dt} + k_p x + F_y = 0$$
⁽²⁾

where m is the mass (N), c the damping factor, k_p the plastic system stiffness (N/m) as evaluated from pull testing of the tendon, and F_y the overall constant yield force (N) for the displacement of the system, calculated as the addition of the tendon material's yield load and the friction factor (eq. 1).

Assuming a critically damped system, the solution to the second order nonhomogeneous differential equation is:

$$x_g = (C_1 \cdot t + C_2) \cdot e^{-wt} - \frac{F_y}{k_p}$$
(3)

$$w = \sqrt{\frac{\mathbf{k}_{p}}{\mathbf{m}}} \tag{4}$$

$$C_1 = \mathbf{v}_0 + \mathbf{w} \cdot \left(\mathbf{x}_0 + \frac{\mathbf{F}_y}{\mathbf{k}_p}\right) \tag{5}$$

$$C_2 = \mathbf{x}_0 + \frac{\mathbf{F}_y}{\mathbf{k}_p} \tag{6}$$

where x_g (m) is the model's calculated tendon head displacement (3), k_p (N/m) the plastic stiffness of the tendon support system, m is the mass (N), x_0 (m) and v_0 (m/s) the initial displacement and speed respectively, *t* is the time (s) and C_1 and C_2 the particular solution constants.

The force in the tendon F_{bar} (N) can be calculated using: $F_{bar}=x_g \cdot k_p$

(7)

and the potential energy absorption (J) can be calculated as:

$$E_{bar} = \frac{\mathbf{m} \cdot \mathbf{v}_0^2}{2} + \mathbf{m} \cdot \mathbf{g} \cdot \max x \tag{8}$$

where g is the gravitational constant (m/s^2) and max x (m) the maximum displacement calculated.

The work We (J) required to pull out the tendon in impact load can be calculated from the load displacement graph:

We= $\int F dx$

(9)

The variables for the rheological model equations (3–6) are parameters m (N), x_0 (m) and v_0 (m/s) which must be defined by the user. The yield load of the tendon material, consisting of part of the overall yield force value F_y (N), as well as the plastic stiffness of the system k_p (N/m), are factors specific to the tendon support system. They can be drawn preferably from quasi-static pull testing and materials specifications, or alternatively from impact tests results. In the latter case, the plastic impact stiffness can be calculated from the end of the load displacement curve at small impact weight velocity, where the peak may appear if the impact load was sufficiently strong to deform the tendon. In normal conditions, the initial displacement is set to zero and the initial velocity corresponds to the impact velocity or block ejection velocity. The mass corresponds to the impact weight or the tendon burden.

In order to obtain the axial load in the tendon as well as the energy quantity absorbed through the tendon's movement in impact load, one can follow the following steps (the use of a spreadsheet is recommended):

- 1 Define an arbitrary friction force F_f representing the friction losses and other energy dissipation sources. For the first iteration, it is suggested to use the tendon support system's yield load measured with pull tests, if not available the yield load of the tendon material.
- 2 Calculate the axial displacement x_g for different time increments, typically in the order of milliseconds, using equations (1), (3), (4), (5) and (6).
- 3 Calculate the axial force in the tendon F_{bar} using equation (7).
- 4 Construct a load displacement graph for the reaction of the tendon. The graph must be set so that if the axial force in the tendon F_{bar} is not greater than the friction force F_{f} , then the force in the tendon F_{bar} equals the friction force F_{f} .
- 5 Calculate the potential energy dissipated through the tendon E_{bar} using equation (8).
- 6 Calculate the work done during the pull We using equation (9).
- 7 Repeat steps 1 to 6 until the potential energy Ebar is approximately equal to the work We.

For modeling of the MCB system in single impact, the factor k_p can be estimated at 775 kN/m, which corresponds to the possible stiffest response calculated from the quasi-static loading tests (Gaudreau 2004). Higher values of k_p are required to calculate the MCB system's reaction to multiple impacts. The plastic stiffness k_p as measured from impact load-displacement graphs of a number of test specimens falls within the range of 760 to 1050 kN/m. The quasi-static evaluation of k_p from test specimens is in the range of 630 to

775 kN/m. It could be argued that the stiffest potential quasi-static k_p is within the range of the impact plastic stiffness.

The steel yield load has a value of 112 kN for the 17.2 mm diameter MCB. Since the energy balance exercise is an iterative process, any value close to the tendon's yield load is considered satisfactory as a first trial for the friction force.

Figures 8 to 10 show the tendon load evaluation result for a sample having a k_p value of 1050 kN/m, a yield load of 112 kN, and a friction force F_f of 93 kN,

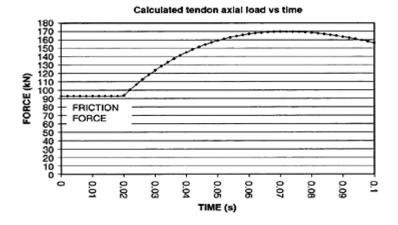


Figure 8. Calculated tendon axial load graph.

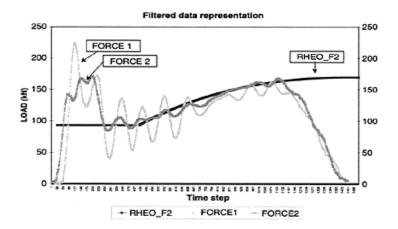


Figure 9. Filtered data representation for instrument signals during test 17-11-2.

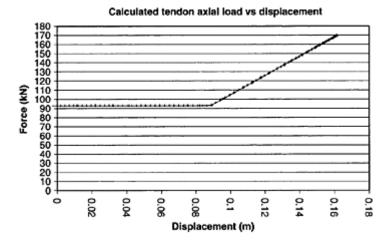


Figure 10. Calculated load-displacement graph.

at an impact weight of 1000 kg and initial impact velocity of 5.7 m/s. This impact velocity corresponds to a drop height of 1.66 m and the k_p value to a tendon whose grout matrix could have stiffened under a first large impact. The maximum load evaluated in the bar is 169 kN (see Figures 8 to 10), and the simulated impact time to the onset of elastic recovery is 70 ms (see Figure 8). On a similar set up, 163 kN was measured for maximum tendon load on sample 17–11–2 (Gaudreau 2004) and the impact duration was 61.2 ms to elactic recovery and 74 ms for the complete test. The friction force of 93 kN falls into the tested laboratory measured range of 80 to 154 kN.

Figure 9 contains a superposition of impact test results for sample number 17–11–2 and that of the proposed calculation model. Force 1 is the measured load on the top of the load frame, Force 2 is the load measured just underneath the reaction plate and Rheo_F2 is the calculated reaction from the model. The model is in good agreement with the experimental data, notably for the calculation of the maximum load in the tendon at the onset of elastic recovery (end of the loading cycle).

The proposed displacement evaluation method for tendons submitted to impact loading can be used to estimate the maximum energy absorption capacity of a given tendon submitted to impact load. The potential energy absorption of a tendon can be estimated by matching the calculated maximum load to the known ultimate tensile capacity of the tendon. The latter can be measured from cyclic impact testing or simply taken from the tendon material's specifications.

For example, if the model is used for a plastic stiffness range corresponding to that of the quasi-static pull out testing results of the program, the calculated single impact absorption capacity of the prototype MCB ranges from 32 to 40 kJ. This result was attained by fixing the impact mass velocity at 5.4 m/s and by increasing the impact weight only to match the ultimate tensile strength of the tendon. With the same range of plastic stiffness and drop weight of 1000 kg, one can calculate the capacity range of the MCB tendon system by modifying the impact speed. The result of such an approach

ranges from 29 to 35 kJ. It appears that there is no particular solution to the energy absorption capacity of a tendon. The latter depends on the combination of ejection speed and tendon burden. Further to this, one can consider that the choice of the potential ejection speed is critical to the evaluation of the level of energy absorption capacity of a tendon. For example, if an ejection velocity of 3.0 m/s is selected, comparable to that selected in the literature review as responsible for the onset of damage in headings, the minimum MCB system capacity becomes 45 kJ. By lowering the plastic stiffness of the bolting system within the range found in quasi-static testing, the bolt reacts entirely in "friction" energy dissipation and its capacity is 50 kJ. The friction energy dissipation in the proposed model equations could include energy losses due to other mechanisms than friction such as heat losses and noise.

After cyclic impact testing to the failure of the tendon, samples 17–11 and 17–14 were sawed apart to

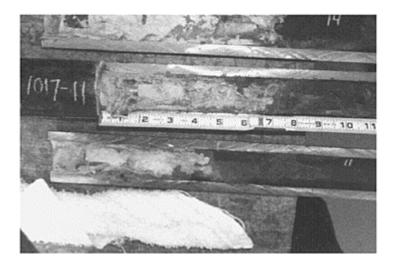


Figure 11. Split test tube for sample 17–11.

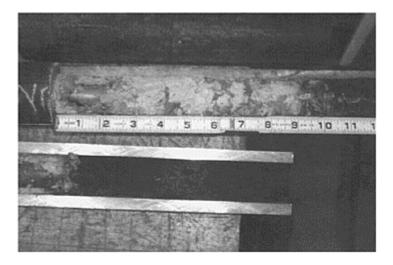


Figure 12. Split test tube for sample 17–14.

measure the amount of displacement of the cone inside the resin. These two samples were chosen because they had been tested using two different drop weights. Figures 11 and 12 illustrate the split test tube samples. Figure 11 shows the cone moved 127 mm into the resin matrix. The drop weight mass used for cyclic impact tests on this sample was 1000 kg. Figure 12 shows the cone moved 177 mm into the resin matrix. The mass used for these cyclic impact tests was 750 kg. The Figures and the proposed calculation method for the tendon displacement under impact loading demonstrate that the prototype MCB both plastically elongates and "plows" through the resin at different degrees depending on the mass-ejection velocity couple.

An application of the proposed displacement evaluation method on cyclic impact testing was considered in the context of this study. However, once the first impact has proceeded, it becomes difficult to evaluate with certainty what the effect of each impact cycle is on the material properties imparted to the model, particularly for the plastic stiffness of the tendon support system. For example, one can refer to the estimated load for the second impact test on sample 17–12 shown in Figures 8 to 10 where the plastic stiffness of the support reaction was chosen to be the maximum plastic impact stiffness measured for all tests.

It is clear that the choice of tendon burden and impact velocity has the greatest influence on the overall calculated or estimated performance of tendon support under impact loading. The phenomenon is accentuated when using small impact velocities or small drop weights. According to the modeled response, more energy could be dispersed by other means than plastic elongation. Note that steel also exhibits non-linear plastic deformation characteristics under impact load (e.g. Ansell 1999, Figure 9). This is not accounted for in the proposed calculation method.

There is no consideration in the calculation method for the adhesion of the tendon inside a holding matrix. The overall work required to debond the tendon from the holding matrix could be added to the work required to pull it in eq. (8). A greater adhesion would increase the energy absorption capacity of the tendon but could reduce its ability to slide. Thus, it was not incorporated into the proposed calculation method.

5 DISCUSSION AND CONCLUSION

In essence, simple pull out testing of tendon support could be envisioned as a means of constructing a set of observations that can be used to approximate the reaction of tendon support in impact load given a simple calculation method such as that presented in section 4. The impact testing rig at NTC premises was quite useful in understanding controlling parameters for the performance of tendon support submitted to impact loading. Although simplified, the displacement evaluation method for tendons submitted to impact loading could be an asset to the engineer wishing to postulate on the best choice of a support system for a given failure mechanism. The model is valid for a velocity range corresponding to that of plausible rockbursts.

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Performance of rockburst support systems in Canadian mines

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ABSTRACT: The resin-grouted Modified ConeBolt (MCB) and the Rockburst Support System were designed for conditions at Noranda's Brunswick Mine where they were introduced successfully in 2001. Since then, the MCB and the Rockburst Support System have been applied at other Canadian operations and a growing number of case studies are becoming available from applications in various loading and rock mass conditions. Examples are given of the performance of the Rockburst Support System under high impact, low-impact repetitive loading and static loading, in hard brittle and weak rock. The ability of the retaining elements (mesh straps, mesh and plates) to transfer the load and mobilize yielding of the MCB tendons is a key component of the success of the Rockburst Support System. Since 2003, research on impact loading of tendons and support systems has been transferred to CANMET-MMSL with the continuation of tendon testing and the development of a new panel testing facility in 2004.

1 INTRODUCTION

1.1 Background

A long-term research project between Brunswick Mine and the Noranda Technology Centre (NTC) was started in 1996 to develop a support system capable of withstanding rockbursts. A dynamic tendon testing facility was built and a comprehensive laboratorytesting program (Gaudreau and Basque, 1999, Gaudreau, 2000a, 2000b) led to the development of a resin-grouted, modified conebolt (MCB) that yields under dynamic loading. This new tendon was combined with retaining elements (heavy gage mesh straps and chain link mesh) to form a Rockburst Support System. Following a series of rockbursts at Brunswick Mine, the first installation of the system began in August 2000 (Simser 2001, 2002a).

The Rockburst Support System was developed for use under Brunswick Mine's rock mass conditions (massive sulphides S.G.=4.3) and targeted to withstand 40 kJ/m² in a single impact. The system consists of a 1 m by 1 m pattern of 2.3 m long modified conebolts (MCB) with 150 by 150 mm, 9.5 mm thick domed plates connected by 2.1 m by 0.3 m #0 gauge weld mesh-straps (strand diameter 7.7 mm) and #6 gauge (4.9 mm strand diameter), 50 mm aperture chain link. The events leading to the design and application of the Rockbust Support System as well as the initial results at Brunswick Mine are described extensively in Simser et al. (2002a, 2000b).

1.2 Applications

Since the initial successes at Brunswick Mine, the Rockburst Support System has been applied in Canadian operations with some modifications. The results to date have been largely positive and have opened up new possibilities in ground support design and applications.

In 2003, the tendon-testing equipment was transferred to CANMET-MMSL's Ottawa laboratory where a research program has been initiated. A complementary panel-testing facility is being developed at CANMET-MMSL's Sudbury facilities for research on ground support systems.

2 CANADIAN EXPERIENCE WITH ROCKBURST SUPPORT SYSTEMS

The first installations at Brunswick Mine called for the use of rolls of chain link mesh to ensure sufficient retaining capacity and minimum overlap. In order to improve productivity, the chain link mesh was later replaced with sheets of welded mesh. Large bulking in the chain link mesh was observed between the conebolts and straps after rockbursting and it was postulated that a stiffer retaining fabric (weld mesh) would be better able to mobilize yielding of the MCBs. This argument seemed to be corroborated by dynamic panel testing of welded versus chain link mesh with mesh straps. The tests were carried out in 2001 by SRK in South Africa (Ortlepp and Swart, 2002). Issues of overlap between sheets were recognized and addressed in the majority of cases by a strict quality control of installation. Full-scale panel static tests (Kuijpers et al. 2002), later indicated that contrary to previous results, welded mesh is not necessarily stiffer than chain link. However, given the productivity improvements realized and the performances observed, most operations have adopted the welded mesh except for particular situations.

The Rockburst Support System with welded mesh was installed at one operation to address three types of situations: high impact loading in fresh (unaltered) brittle rhyolite, low impact, repetitive loading in massive sulphides and slow deformation in weak altered rhyolite and talc. The support system has performed very well in all three situations.

2.1 High impact loading

In at least three distinct situations it was possible to compare the performance of standard support (rebar and screen or rebar and mesh-reinforced shotcrete) with that of the Rockburst Support System. Two examples are presented here. Figure 1 shows the damage observed in fresh brittle unaltered rhyolite after an event of Moment Magnitude=1.4 located 54 m away. The start of damage coincides with the end of the Rockburst Support System coverage. The latter had been installed specifically to reinforce the intersection from which the photo is taken. Due to the size of the intersection (>8m span) and the heavy traffic, 7 m cables,

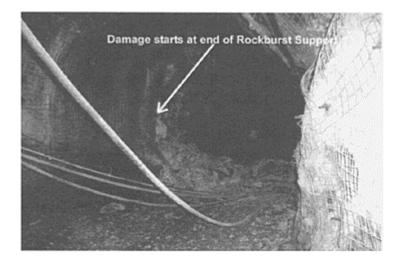


Figure 1. Case study 1 shows an example of the performance of the Rockburst Support System under high impact loading.

de-bonded over a length of 3.7 m had been added to the Rockburst Support System as shown in Figure 2.

Following the event, it was observed that two cable plates had been ripped out due to failure of the cable grips, one MCB had been pushed out probably indicating that it was broken, and the support fabric was stretched and deformed on the walls, but it did not fail. Drilling confirmed a depth 1.2 to 2.4 m of fracturing behind the support. Overall, the Rockburst Support System responded extremely well and the intersection was operational immediately after the event. By comparison, two sections of drift (4.5 m span) located 10 and 20 m away from the intersection, as shown in Figure 1, and supported with mesh-reinforced shotcrete and rebar failed, as can be seen in Figure 3, and required extensive rehabilitation.

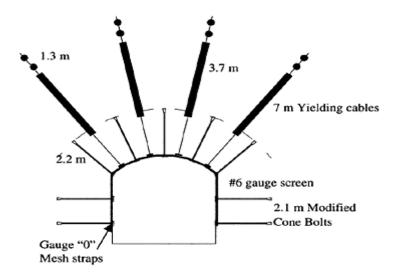


Figure 2. The Rockburst Support System with 7 m debonded cables for large intersections and future stope backs.

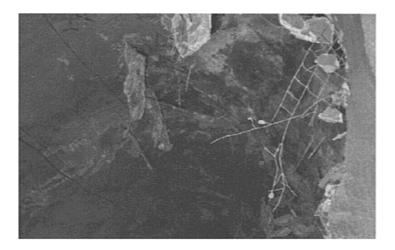


Figure 3. The mesh-reinforced shotcrete and rebar could not sustain the impact from the event. The rock mass disintegrated around the rebars and the full capacity of the tendons was not mobilized.

Figure 4 shows Case study 2 in fresh brittle unaltered rhyolite where the Rockburst Support System helped minimize the damage and allowed a rapid return to normal following a Moment Magnitude=1.1 event located 38 m from the observed damage. The damage (ejection of rock) was confined to areas where the support had not been upgraded to the Rockburst Support System. Drilling indicated fracturing in the back and southern wall of the excavation up to a depth of 2.4 to 3 m behind the support, yet the only ejected material came from the small bay with no Rockburst Support.

2.2 Low impact repetitive loading

A shrinking pillar between two retreating pyramids was considered to be at high risk for rockbursting due to the progressively increasing stress regime and the presence of a rolling sub-horizontal dyke cross-cutting the zone (Falmagne et al. 2004). The Rockburst Support System with de-bonded cables shown in Figure 2 was installed in the main accesses to the zone that had to be operational throughout the life of the zone.

As mining progressed and the area became more highly stressed, a large number of small magnitude seismic events were recorded in the zone, particularly around the folded dyke. Instrumentation has indicated 37 mm and 67 mm of displacement in the backs of a drift and intersection. Fracturing and bulking of the walls behind the welded mesh screen and flattening of the domed plates confirm the instrumentation readings but there has been no failure of the support and the accesses have fulfilled their purpose without any rehabilitation delays.

In another operation, the Rockburst Support System has been very successful in containing the deterioration of accesses in secondary pillars located in a sill pillar. Previous experience had shown that extensive



Figure 4. Case study 2 is another example of the effectiveness of the Rockburst Support System at retaining and holding fractured rock in place. The depth of fractured rock is deeper than the length of the support. deterioration of the accesses occurred during mining of the primary stopes.

In both operations, the use of the Rockburst Support System in a low impact repetitive loading environment enabled the operators to proceed with mining in the zone without any interruptions or unplanned delays, allowing the operators to meet and even exceed the mine's planned production schedule.

2.3 Static loading, high deformation

In two areas, the Rockburst Support System was installed to support very weak rock; namely, highly altered rhyolite and a talc band. Other types of ground support had limited success in controlling the deformation in these areas and it was decided to test the yielding capability of the MCB in squeezing ground.

Figure 5 shows the ability of the MCBs and support system to accommodate the deformation around the talc band while plates were ripped off the rebars, and shotcrete cracked extensively behind the welded mesh. The talc band eventually failed during mucking of the last stope in the pillar. Measurements indicated up to 200 mm of convergence prior to mining the stope and it is expected that the displacement values would have been significantly higher if they had been taken from the time of installation up to failure. Nonetheless, assuming an even distribution of displacement on both walls, it can be concluded with certainty that the MCBs can accommodate at least up to 100 mm of displacement under quasi-static loading in-situ before failure.

In Case study 4, the Rockburst Support System has been subjected to quasi-static loading followed by impact loading from seismic events located in the fresh rhyolite behind the altered rock mass supported by the Rockburst Support System. Figure 6 shows the bulking of the wall with one potentially broken MCB

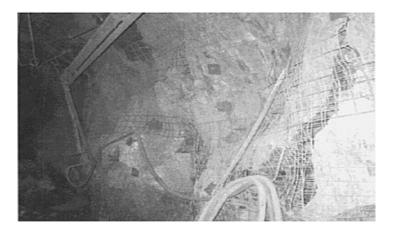


Figure 5. Case study 3 indicates that the Rockburst Support System is able to retain and hold a weak rock mass with deformations of at least 100 mm.



Figure 6. Case study 4 shows the performance of the support system in highly altered and weak rhyolite subjected to both quasi-static and low impact dynamic loading. A sheared MCB can be seen in the foreground.

following the seismic activity. The Rockburst Support System has performed better under static loading than any other support type previously installed in this area and it has demonstrated the ability to sustain low impact dynamic loading following an episode of deformation.

2.4 Non-axial loading and binding of the cone

Under dynamic loading, the MCB dissipates energy by plowing through the resin column that is pulverized and flows around the cone. If, at any point, the cone is prevented from moving, the tendon steel will stretch and eventually break. This mechanism was envisaged for the behavior of the MCB under non-axial loading and at least two case studies have confirmed that the cone plow mechanism is ineffective under these conditions.

In one case, a large seismic event caused a shift of the stress-fractured rock above an intersection thus preventing the MCBs from yielding (cone plow). In this situation, a longer support such as de-bonded cables and a more forgiving retaining element such as chain link mesh may have helped maintain the fractured rock mass in place.

At another operation, it is observed that the deformation in secondary pillars is mainly accommodated by sliding along distinct joint sets and shear zones oriented at approximately 45° to the centre line of the drift. The MCBs installed across the sliding structures have been failing systematically around the joint sets, as the bolts are not loaded axially.

At a third site, bulking of the rock mass, under dynamic loading without yielding of the tendon due to shear displacement, has been observed to cause local failure of the #0 gauge (7.7 mm strands) mesh-strap and ejection of fractured rock. These observations from three different mining operations and rock types confirm the prediction that non-axial loading of the MCB may bind the cone and prevent it from plowing through the resin. In this situation, the energy absorption capacity of the MCB thus depends mainly on the steel capacity to absorb the deformation. Laboratory tests have provided some insight on the behavior of the MCB under these conditions and will be discussed in the following section.

3 LABORATORY TESTING

The ability of the MCB to withstand repeated impacts of approximately 15 kJ has been well established in the laboratory (Gaudreau 2000a, 2000b), and although limited measurements are available in-situ, the application of the Rockburst Support System has repeatedly demonstrated the superiority of the MCB over non-yielding tendons. The maximum single impact that can be sustained by one tendon however, has only been deduced so far from the results of static testing, multiple impact loading and modeling (Kaiser 2001; Gaudreau 2000b) and remains to be verified in the laboratory. Instrumented tests that track the movement of the cone during static and impact loading provide additional insight on the behavior of the tendon-resin-steel tube system.

3.1 Static testing

Static pull tests have been performed in the laboratory and in-situ, and are presented in Figure 7. The laboratory test ("static test") was performed on a 2.1 m long MCB under the same conditions (resin and mixing) as the in-situ test. The "static test" curve shows that the tendon started to yield at approximately 9.7 tonnes and broke at the threads after maintaining a load of 17–18 tonnes and sustaining over 200 mm of displacement Monitoring of the cone displacement during the "static test" indicated that the cone moved a total of 23 mm as shown in Figure 8 before failing at the threads.

In-situ tests can be grouped in two categories: no cone movement and limited cone movement comparable to laboratory results. The first series of tests (BM bolt 2, 4, 5, 6 suplh and BMbolt 3 seds) closely mirrors the standard steel curve provided by the manufacturer. In this case, it is doubtful that any cone movement occurred. The second series of tests (BAS/N 7–2, BA#3 and BA#1) follows the laboratory test (static test) curve. It is therefore concluded that some cone movement, of the order of 23 mm probably occurred, thus softening the system by comparison with the steel alone. The observations and in-situ measurements described in the previous section indicate that efficient support systems for quasi-static or slow loading situations may be designed around the MCB as long as the limitations of the tendon are recognized, taken into account and deformations are monitored.

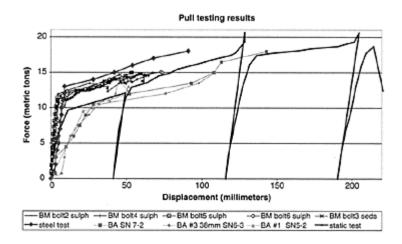


Figure 7. Laboratory and in-situ static testing of MCB.

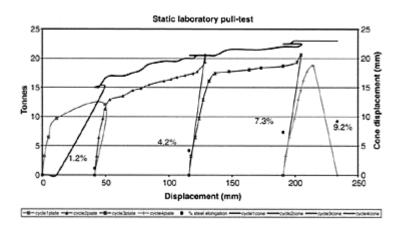


Figure 8. Laboratory pull-test of a 2.1 m MCB. The movement of the cone was monitored throughout the test and the average steel elongation is calculated from the difference between the plate and the cone movement.

3.2 Repeated impact testing

A fundamental difference between the MCB and the original South African conebolt is the fact that the latter was designed to avoid exceeding the steel yield capacity at all times. With the resin-grouted MCB under multiple impacts, the cone displacement absorbs most (~80%) but not all of the energy. Several instrumented impact tests have shown that even for the same sample, the steel occasionally absorbs the majority of the energy for any particular impact. Table 1 shows the results for two samples that took respectively 7 hits before breaking or hitting the floor.

The cone movement was monitored and the average steel elongation is calculated as the difference between the plate and cone displacement. It can be seen that in at least one instance for each sample (sample 1: drop 4, sample 2: drops 5 and 7), the cone did not move and the entire impact energy was absorbed by the steel and possibly dissipated as friction along the bar. While not desirable, this demonstrates that the MCB may be able to survive 15 kJ impacts even in less than ideal loading situations provided that the steel has not reached its ultimate capacity. The life of the tendon is however compromised as testing shows

Drop no	Ep (kJ)	Plate displ. (m)	Cone displ. (m)	Steel Stretch (m)	% cone disp.	Cum. % steel elong.
Sample	Sample 1					
1	13.0	0.0809	0.0143	0.0666	17.7	2.9
2	13.63	0.1318	0.1222	0.0096	92.7	3.3
3	14.59	0.1365	0.1365	0	100	3.3
4	15.74	0.0762	0.0008	0.0754	1.0	6.6
5	16.98	0.1334	0.1254	0.008	94.0	7.0
6	18.09	0.254	0.2445	0.0095	96.3	7.4
7	21.85	Hit floor	0.3413	Hit floor		
Total			0.985			
Sample	2					
1	12.71	0.1969	0.137	0.0599	69.6	2.6
2	14.75	0.1793	0.1762	0.0031	98.3	2.8
3	16.40	0.1763	0.1698	0.0065	96.3	3.0
4	18.22	0.1793	0.162	0.0173	90.4	3.8
5	20.58	0.1111	0	0.1111	0	8.7
6	21.87	0.1651	0.154	0.0111	93.3	9.1
7	23.44	Fail	0	Fail	0	
Total		1.008	0.799			

Table 1. Multiple impact tests on MCB.

(After Simser 2002.)

that the ultimate steel elongation capability lies between 7 and 9% elongation, somewhat less than the steel alone but comparable to the laboratory pull-test discussed above (static test).

4 DESIGN OF SUPPORT SYSTEMS

The three primary functions of support elements are defined in Kaiser et al. (1996) as Reinforce, Retain and Hold. Under dynamic loading it is necessary to ensure that the support system includes both Yielding elements and a Retaining element capable of transmitting the impact load to the tendons ("snow-shoe" effect) and mobilizing their yield capability. In the Rockburst Support System, the mesh-straps and plates act as the "tendon mobilizers" and play a key role in the success of the system.

Observations indicate that the most effective system is achieved when the Rockburst Support System (weld mesh, MCBs and mesh straps) is installed over mesh or fibrereinforced shotcrete. The additional strength, stiffness and smoothness of the retaining element (mesh over shotcrete) help spread the load more evenly to the tendons and mobilize yielding of the MCBs. The shotcrete alone is brittle and does not resist high impact loading; however, when covered by screen and mesh straps, it retains its usefulness even if it is heavily broken. This combination is expensive and may not be practical, but it helps demonstrate the necessity for careful design of the retaining element that must be neither too stiff (or it may break between the mesh straps), nor too soft (as it may not mobilize the tendons).

In large intersections (>8 m span) additional Holding capacity should be added to the Rockburst Support System to maintain the heavily fractured rock in place following an event. Some success has been achieved with de-bonded cables in this situation.

4.1 Ground support research at CANMET-MMSL

The impact-testing machine for tendons was transferred from the Noranda Technology Centre to CANMET-MMSL's laboratory facilities in Ottawa (Bells Corners) in 2003. Since then some improvements have been made to the instrumentation and data acquisition system and the 2004 testing program plans to address the following issues:

- 1 The maximum single impact capacity of MCBs.
- 2 The effect of drill hole diameter on the performance of MCBs.
- 3 The performance of other types of tendons (cables) under impact loading.
- 4 In-situ monitoring of cone displacement and steel elongation.

Although much has been learned from the impact testing of tendons in the laboratory, questions remain about the behavior of a complete support system under impact. Facilities for panel testing have been built in South Africa and in Australia and they have proven very useful to the respective mining industry and research community. A proposal is thus underway to build a panel testing facility at the CANMET-MMSL facility in Sudbury, Ontario in 2004.

5 CONCLUSIONS

Based on the original research on yielding tendons in South Africa, a resin-grouted modified conebolt (MCB) was developed and tested by Noranda Inc. between 1996 and 2002. In 1999, a Rockburst Support System incorporating the newly developed MCB, #0 gauge mesh straps and chain link mesh was developed for Brunswick Mine conditions and targeted to withstand approximately 40 kJ/m². Experience has since shown that the yielding tendons perform well under impact loading provided that the retaining elements of the support system adequately transfer the load to the tendons. In several Canadian mines heavy gage mesh straps have been successful at transferring dynamic loads to the MCB, as well as connecting neighboring bolts and creating an effective support system.

The Rockburst Support System has been successful in Canadian operations and a number of case studies are now available to evaluate its performance under different loading (high impact, low impact repetitive and static loading) and rock mass (hard, brittle and weak) conditions. A growing database of laboratory and in-situ testing and performance is becoming available from a number of Canadian operations and will contribute to the evolution of support systems customized to different environments. Research and testing of support systems and their individual components is being pursued at CANMET-MMSL's facilities in collaboration with the Canadian mining industry and manufacturers.

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Assessing the *in-situ* performance of ground support systems subjected to dynamic loading

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ABSTRACT: In the past, simulated rockbursts using blasting have been carried out to assess the relative performance of bonded surface support systems (Espley et al. 2002, Archibald et al. 2003), study the response of rockbolts to dynamic loading (Haile et al. 2001, Tannant et al. 1994 and 1995) and for broad ground motion studies (Hagan et al. 2001). Simulated rockbursts can be used to assess the performance of complete ground support systems *in-situ* when subjected to strong ground motion due mining induced seismicity and rockbursting. A series of such experiments has commenced at a number of seismically active Western Australian underground mines, with the aim of testing the *in-situ* performance of a number of standard and innovative ground support systems under strong dynamic loading conditions. The testing includes ground motion measurement with a 16 channel Impulse seismic monitoring system, three-dimensional photogrammetric imaging using the CSIRO developed Sirovision package, crack monitoring using a borehole camera, digital video camera filming and extensive manual measurements and mapping. This paper discusses the methodology involved in carrying out these simulated rockbursts and also presents the issues that will be addressed with the view to ultimately improve our understanding of ground support system performance under strong dynamic loading conditions.

1 INTRODUCTION

As Western Australian underground mines reach increasingly greater depths, the problems of mining induced seismicity and rockbursting have necessitated the use of ground support which is capable of withstanding strong dynamic loads. There is a clear need to optimize the design of such dynamic support systems in mines through developing a better understanding of their behaviour under strong dynamic loading.

Previous ground support testing programs involving the use of drop weights or laboratory simulations have provided important information on the load-deformation characteristics of individual support elements under dynamic loading conditions. These tests do not, however, account for rock-support interaction or the influence of local rockmass conditions. The study of *in-situ* rockburst damage can only be carried out after the fact. When relying on such data, researchers have no control over the location and nature of the seismic source, often leading to ambiguous results. By simulating rockburst damage using blasting, it is possible to investigate the *in-situ* performance of complete ground support systems (incorporating reinforcing, retaining and surface support elements) due to a range of measurable dynamic loads.

By simulating rockbursts at several mines, the research will investigate the influence of rockmass discontinuities, rockmass damage, and stress conditions on rockburst damage, as well as the influence of support and reinforcement systems to reduce displacement and damage due to dynamic loads. Some specific issues which can be addressed at individual mines include:

- How effective are a mine's standard support systems under dynamic loading conditions?
- What is the effect of support and rockmass deterioration on the performance of the support system?
- Is standard Western Australian support (e.g. Split Sets) sufficient if the support density is increased, coupled with strong surface support (e.g. mesh reinforced shotcrete)?
- At what point (or level of dynamic loading) are yielding reinforcing elements required (e.g. Cone Bolts and debonded cables)?
- Is mesh strapping a suitable substitute for shotcrete?
- What is the effect of bolt spacing on support system performance?
- What amount of deformation is permitted for the support system to remain functional?
- Could thin spray-on liner products provide cost and safety improvements in seismically active Western Australian mines?

Combined with the results of drop weight testing, case studies and laboratory simulations, this research will contribute to the development of more robust ground support design procedures, leading to improvements in safety and cost efficiency in areas of underground mines affected by rockbursting.

At the time of writing, analysis for the first series of simulated rockbursts is underway—undertaken at the Long Shaft mine in Kambalda, WA.

2 TESTING METHODOLOGY

2.1 Test layout

The simulated rockburst experiments are conducted by blasting adjacent to the walls of disused excavations. Test sites must be located near an intersection to allow drilling of blastholes parallel to the test wall. The excavations chosen are preferably located in highly stressed areas of the mine, where pre-existing stress driven rockmass damage surrounding excavations will allow for large dynamic loading upon ground support systems subjected to strong ground motion. Typically, three blastholes are drilled parallel to and 5 m from the test wall at each site, as shown in Figures 1 and 2. A distance of 5 m is used to attempt to limit the influence of explosive gases generated during blasting on the test wall (Hagan *et al.* 2001).

Each blasthole is separately charged and detonated to allow successively larger dynamic loading upon the test wall. The central hole is used firstly to generate a small calibration blast using a single primer (to ensure the instrumentation is functional) then charged again (the first simulated rockburst) with an aim to generate a peak particle velocity (PPV) of up to 0.5 m/s. The top blasthole (the second simulated rockburst) is charged to generate a PPV of up to 1.5 m/s on the test wall. The bottom hole (the third simulated rockburst) is charged with a view to achieve a PPV of up to 5 m/s. Further particulars of the blasting are discussed in Section 2.4.

Series of simulated rockbursts will be undertaken at a few sites within each mine to allow testing of a range of ground support systems under varied rockmass conditions. Freshly and previously installed ground support systems will be tested at different sites, to investigate the effect of loss of support functionality due to deterioration from corrosion, past dynamic loading and deteriorated ground conditions. For freshly installed sites, two ground support systems are installed side by side to allow direct physical

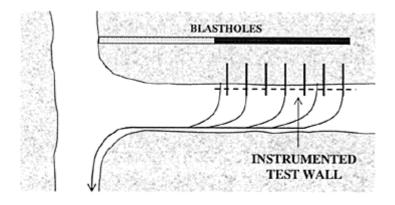


Figure 1. Conceptual layout of simulated rockburst experiments.

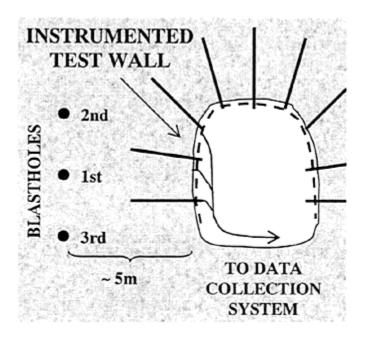
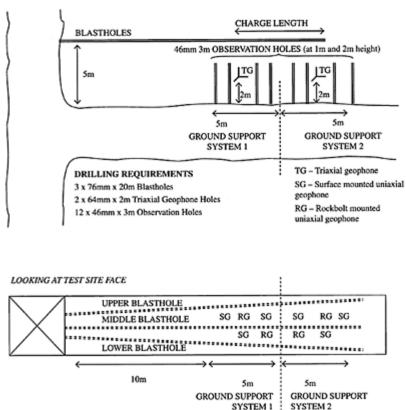


Figure 2. Conceptual cross-sectional view of simulated rockburst experiments.

comparison in the same rockmass conditions (see Figure 3). A 5 m length of each support system is installed, giving the test wall a total length of 10 m.

2.2 Instrumentation and equipment

Ground motion monitoring at each site is conducted using 14 Hz SM6 geophones connected to a 16 channel Impulse seismic monitoring system. The Impulse allows a maximum sampling rate of 10 kHz per channel, which is adequate to prevent aliasing of waveforms, considering the maximum frequency of ground motion expected to be generated by the blasting is not more than 1 kHz. Two triaxial geophones are installed in 2 m long 64 mm diameter blastholes, drilled perpendicular to the test wall. The triaxial geophones provide an indication of the characteristics of the ground motion generated by each blast, as well as allow comparison between the PPV in the intact rock to the PPV at the



PLAN VIEW OF TEST SITE

Figure 3. Conceptual layout of a typical simulated rockburst site showing drilling requirement and approximate sensor locations.

surface of the excavation. A further 6 horizontal uniaxial geophones are mounted on the surface of the test wall. The remaining 4 channels are used for horizontal uniaxial geophones end-mounted on rockbolts. It is envisaged that data from these rockbolt geophones will provide insight into the dynamic response of the different rockbolt types as well as estimates of the degree of interaction between these elements and the rockmass. All geophones are mounted using two-part epoxy resin. Protection of cables during blasting is important for the successful use of the monitoring system for simulated rockbursts. Experience from the first set of simulated rockbursts, however, indicates that while some cables can be expected to be severed after each blast, transmission of the shock wave data occurs before breakage, allowing waveform data to be recorded.

As the maximum input voltage of the Impulse seismic monitoring system is 10 V/(m/s), and the response of the SM6 geophones is approximately 28 V/(m/s), some

degree of geophone damping is required to prevent 'clipping' of waveforms during blasting. By incorporating a $25-30\Omega$ shunt resistor into each geophone circuit, the geophone response can be altered to around 2 V/(m/s), which allows the Impulse to record ground motions of up to around 5 m/s, suitable for the largest blast. The maximum coil excursion for the SM6 geophones (4 mm) was not exceeded during the first series of simulated rockbursts.

A number of 46 mm observation holes are drilled perpendicular to the test wall for borehole camera observations. These measurements allow investigation of the degree and nature of rockmass fracturing before and after each blast. The borehole camera is also used inside Split Sets (which are used extensively in the mine sites at which testing will be undertaken) to attempt to identify slip or plastic yield along their length.

Extensive mapping of each test site before and after each blast is carried out using Sirovision, a three dimensional photogrammetry system developed by the CSIRO and designed primarily for structural mapping of underground or open cut rock faces (CSIRO 2003). Sirovision allows the generation of fully digitised three dimensional images from pairs of photographs, when accurate survey support is available. As well as mapping, images generated before and after successive blasts are used to identify areas of rock bulking or ejection, accurately measure deformation of surface support and measure the displacement of rockbolt ends. These measurements contribute greatly to the qualitative and quantitative assessment of ground support system performance.

All simulated rockbursts are recorded using a Canon MV750i digital video camera, set up at sufficient distance from the test wall to avoid damage due to ground shaking or ejected rock or support fragments. The camera is used to maintain a permanent record of each simulated rockburst, as well as assessment of the nature and velocity of ejected rock and support fragments during the experiments. Rockmass damage due to the simulated rockburst occurs before dust obscures the video record.

A conceptual layout of a simulated rockburst test wall is shown in Figure 3 at the end of the paper. The Figure shows 8 pairs of observation holes located at 1 m and 2 m height from the tunnel floor, located in line with ground support rings. The surface geophones are shown to be distributed evenly over the test wall. Triaxial geophones are located centrally in each ground support regime, within the range of heights of the observation holes.

2.3 Measurements and observations

A combination of the methods described in the previous section and manual techniques is used to assess the performance of each ground support system when subjected to each successively larger dynamic load. Each section below summarises the measurements and observations taken as well as the methods used.

2.3.1 Geotechnical properties of the test site

Conventional methods such as geotechnical face mapping of the test wall and nearby stress measurements are used. Additionally, CSIRO Sirovision is used to provide a photographic record of the test site at all stages of blasting and also to allow analysis of rockmass structure using the Sirojoint module. This module allows the user to accurately measure the parameters defining discontinuities in a rockmass and analyse the structure of a rockmass by using three-dimensional images (CSIRO, 2003).

2.3.2 Ground motion

Peak particle velocity is measured using both surface mounted uniaxial geophones and borehole mounted triaxial geophones. The Impulse seismic monitoring system software is used to download raw waveform data from the unit, generate waveforms based on the damping parameters of the geophones and for exporting the processed waveforms for further analysis.

2.3.3 Location and behaviour of rock fractures

A borehole camera is used before and after each successive blast to locate existing fractures or jointing and then to assess the influence of the dynamic load on rockmass behaviour. It is considered that measurement of the location of new fractures or the behaviour of existing fractures, together with accurate measurement of ground motion, can allow the estimation of dynamic load (in terms of kJ/m^2) on the ground support system.

2.3.4 Identification and accurate measurement of areas of rock bulking and ejection

Analysis of the three dimensional images generated by Sirovision allows the determination of volumes of ejected or bulked rock over the entire test wall. A comparison of the digital models created by the software before and after each successive blast also aids greatly in quantitative damage assessment. The high resolution digital camera used in these experiments (Nikon D100) allows generation of extremely detailed three dimensional images which provide a permanent record of the test site following each simulated rockburst.

2.3.5 Measurement of ejection velocity and maximum displacement of rochnass and support

Three methods are available for these measurements. Firstly, calculation of ejection velocity is possible from digital video camera images, provided the position and orientation of the camera are considered. One problem with this method is that the camera recording frame rate must be lowered if lighting is inadequate, which is often the case in underground mines. Secondly, maximum displacement of the test wall surface or monitored rockbolts may be estimated from integration of velocity waveforms. The final method involves back calculating projectile ejection velocity based on it final position on the test site floor (ballistics), however this is likely to be inaccurate.

2.3.6 Source parameters of simulated rockbursts

All mines at which simulated rockburst testing will be undertaken have mine-wide seismic monitoring systems, which can be used to assess the source parameters of each simulated rockburst blast. Seismic sensor saturation on the mine-wide seismic system was found to be an issue at the first test site. This affects source parameters such as energy release and magnitude, however if enough sufficiently distant sensors record the blasting, these issues can be managed.

2.3.7 Response of reinforcing and retaining elements to dynamic loading

Uniaxial geophones end-mounted on rockbolts allow direct measurement of peak particle velocity induced in the rockbolt steel as a result of the dynamic load caused by blasting. This allows estimates of the degree of rockbolt coupling with the rockmass, the dynamic stress induced in the rockbolt and analysis of the frequency response of the rockbolt. The response of Split Set friction bolts can also be examined through the use of a borehole camera, whereby failure or yield of the Split Set can potentially be assessed in terms of the distance slipped, the location of rupture or (although much harder to identiiy) plastic elongation.

2.3.8 Response of surface support elements (shotcrete, mesh and thin spray on liner) to dynamic loading

Again, Sirovision can be used to calculate the permanent deformation of surface support elements through comparison of three dimensional point locations on the test wall before and after each successive blast. Also, detailed fracture mapping of shotcrete is undertaken as well as qualitative measurement of adhesion by assessing shotcrete 'druminess'.

2.3.9 Assessment of loss of functionality of the support system

Qualitative assessment of the *in-situ* performance of each ground support system is based upon the degree of remaining functionality of the system, following each blast. This approach was employed during the Geomechanics Research Centre's Canadian Rockburst Research Program (Tannant & McDowell, 1995). The Support Damage Scale (SDS—see Table 4 at the end of the paper) was used during this research to assess the performance of installed ground support following damaging rockbursts. It is envisaged that a similar scale will be used in this project, altered to suit the ground support elements used at each mine site where testing is conducted.

Because support damage varies over the supported area, distributions of ground support performance can be generated by assessing the SDS rating at grid points over the test wall for which a PPV is known. PPV contouring between sensors is used to estimate a ground motion value at these grid points. This approach was employed successfully to analyse data from previous simulated rockburst trials on bonded support systems at INCO in Canada (Espley *et al.* 2002).

2.4 Blasting details

The simulated rockburst blasts are designed to reproduce the dynamic loading associated with a seismic event. This can be achieved by maximising the release of shock wave energy and minimising the effects of rapidly expanding gases.

Also, whilst blasts don't typically generate a significant shear wave, the chances of generating one improve with the use of an explosive for which the velocity of detonation (VOD) is less than the P-wave propagation velocity in the surrounding rock (Hildyard & Milev, 2001). However, low VOD explosives typically release more energy through the rapid expansion of gas than through shock effects. For these reasons, emulsion products are considered to be the most suitable blasting agent. It is preferable to not use ANFO for the simulated rockburst experiments as the effects of rapidly expanding blast gasses may result in the ejection of rock and support materials from the test wall, potentially compromising the test results. Ultimately, however, the explosive used for the experiments is often dependant on what products are available at each mine site.

Details of the blasts conducted at simulated rockburst sites are presented in Tables 1,2 and 3. The tables show approximate charge lengths and mass of explosive required in a 76 mm borehole for a pumped emulsion, a packaged emulsion and ANFO. These types of explosives, or similar, are considered to be the most commonly available explosive types in Western

	Blast number			
	Calibration	1	2	3
Location	Mid height	Mid height	Upper	Lower
Target PPV (m/s)	~0.01	0.5	1.5	5
Approx. mass of pumped emulsion required (kg)	0.25 kg primer	8	16	44
Approximate charge length required (m) (76 mm borehole)	0.25 kg primer	1	3	8

Table 1. Approximate charge details for simulated rockbursts using pumped emulsion (Orica 2500 Powerbulk Emulsion).

Table 2. Approximate charge details for simulated rockbursts using packaged emulsion (Dyno Nobel Powermite Advance 65×400 Cartridges).

		Blast number		
	Calibration	1	2	3
Location	Mid height	Mid height	Upper	Lower

Target PPV (m/s)	-0.01	0.5	1.5	5
Approx. mass of packaged emulsion required (kg)	0.25 kg primer	8	18	63
Approximate charge length required (m) (76 mm borehole)	0.25 kg primer	1.5	3.5	12

Table 3. Approximate charge details for simulated rockbursts using ANFO.

	Blast number			
	Calibration	1	2	3
Location	Mid height	Mid height	Upper	Lower
Target PPV (m/s)	-0.01	0.5	1.5	5
Approx. mass of packaged emulsion required (kg)	0.25 kg primer	7	18	73
Approximate charge length required (m) (76 mm borehole)	0.25 kg primer	2	5	20

Australian mines. The maximum PPV in these tables were estimated based on the equations presented in Ouchterlony (1993):

$$\mathbf{v}_{\max} = \mathbf{a}_{1} \left(\frac{R}{\sqrt{f_{1} \left(\frac{R}{L_{\epsilon}} \right) \mathcal{Q}}} \right)^{-b_{2}}$$

where the function:

$$f_{l}\left(\frac{R}{L_{e}}\right) = \frac{\arctan(L_{e}/2R)}{L_{e}/2R}$$

where:

v_{max} is the maximum expected PPV (mm/s);

R is the distance from the source (m);

Q is the charge weight (kg);

L_e is the charge length (m); and

a1 and b2 are mine dependent constants related to rockmass attenuation.

Also, if the diameter of the charge is smaller than the diameter of the charge hole, the degree of coupling is defined as:

$$f_{eh} = \phi_e / \phi_h$$

where Φ_{e} is the diameter of the charge and Φ_{h} is the diameter of the hole. The corrected velocity is then:

$$v_{max \text{ corrected}} = v_{max} f_{eh}^{1.5}$$

This approach takes into consideration the effects of a linear charge of defined length, the type of explosive used, the rock type, rockmass condition and confinement effects relating the diameter of the blasthole to that of the charge. The two rockmass dependant constants required for this approach (a_1 and b_2) were set at 698 and 0.74 respectively, the same values used by Ouchterlony (1993) for a Precambrian granite, since the first set of simulated rockburst experiments was conducted within felsic intrusions.

Based on these figures, it is apparent that it becomes impractical to use ANFO in a 76 mm hole for the 5 m/s blast, since the charge length becomes greater than the expected length of the test section (10 m). Generating 5 m/s PPV over such a large length of the cross-cut increases the likelihood of severe damage which may restrict re-entry and prevent full analysis of the results. Also, while the distance between the blastholes and the test wall may be decreased to reduce the amount of explosive required, this may cause damage due to gas expansion, which would adversely affect the results.

3 CONCLUSION

Whilst considerable work has gone into the design of the simulated rockburst test program, a number of problems may arise which can potentially affect the experiments. These include:

- Loss of access to the test wall following a blast due to more extensive damage to the tunnel than anticipated;
- Blasting misfire—in which case a blasthole may need to be abandoned;
- Inaccurate surveying of Sirovision control points leading to errors in measurement of angles and distances on digital images;
- Failure to trigger the Impulse monitoring system—at the first simulated rockburst site, concerns regarding this problem led to the removal of resistors from two geophone circuits (and hence an increase in sensitivity) to ensure triggering would occur; and
- Shaking or dust affecting the digital video recording.

Early analysis of data from the first set of simulated rockburst experiments is yielding excellent results, with the only issue of concern being shaking and dust affecting the digital video recordings of the two larger blasts. However constraints on the placement of the camera due to the geometry of the first

]

McDowell, 1995).					
Damage level	General description	Support damage	Shotcrete damage		
S 0	Conditions unchanged	No new damage or loading	No new damage or loading		
S1	Support undamaged but first signs of distress detectable	No damage to any support component	Shotcrete shows new cracks, very fine or widely distributed		
S2	Slight damage to support Loading clearly evident but full functionality maintained	Plates and wooden washers on some rockbolts are deformed, showing loading Individual strands in mesh broken Mesh bagged but retains material well	Shotcrete cracked, minor flakes dislodged Shotcrete is clearly taking load from broken rockmass (mostly drummy)		
S3	Moderate damage to support Support shows significant loading and local loss of functionality; retaining function primarily lost (except in laced or shotcreted areas)	Plates, wooden washers, and wood blocking on rockbolts are heavily deformed, showing significant loading; bolt heads may be 'sucked' into rock Mesh torn near bolt heads with some strands broken and mesh torn or opened at overlapping edges Moderate bagging of mesh and isolated failures of rockbolts Cable lacing performs well	Shotcrete fractured, often debonded from rock and/or reinforcement Major flakes possibly dislodged Holding elements mostly intact		
S4	Substantial damage to support More extensive loss of retaining and holding functions (except for lacing systems)	Mesh is often torn and pulled over rockbolt plates; if it did not fail, it is substantially bagged (at capacity) Many rockbolts failed Rock ejected between support components Cable lacing is heavily loaded with bagged mesh	Shotcrete heavily fractured and broken, often separated from the rockmass with pieces lying on the ground or hanging from reinforcement (Connections to holding elements often failed or holding elements failed locally)		
85	Severe damage to support Support retaining, holding, and reinforcing functions failed	Most ground support components broken or damaged Most rockbolts fail and rock peels off cable bolts	For damage level S5, shotcrete fails to be functional and the lefthand column applies		

Table 4. The Geomechanics Research Centre Support Damage Scale (SDS) (Tannant & McDowell, 1995).

Shotcrete non-functional Mesh without cable lacing heavily torn and damaged Cable lacing systems heavily stressed and often failed

Notes: (1) The damage indicators listed in this table describe damage that is new and was caused by the rockburst. If the observer cannot ascertain that the damage was inflicted by the rockburst then the damage should be ignored for the purposes of damage classification.

(2) One or more damage scales may be observed in same section and should be recorded separately.

(3) Rock and support damage levels need not correspond.

(4) Because the function of shotcrete support is somewhat different and more complex than for other support systems, a separate column of indicators is provided over the range of S0 to S4. It is important to record where shotcrete is present and when it has been used to determine the support damage level.

(5) Failure of rockbolt applies to failure of nut, plate, anchor or shank.

(6) From: Kaiser, P.K., Tannant, D.D., McCreath, D.R. & Jesenak, P. 1992, 'Rockburst damage assessment procedure', Rock Support in Mining and Underground Construction, eds. Kaiser & McCreath, Balkema, Rotterdam, pp. 639–647.

site are considered to have been the main cause for the problem and it is anticipated that future sites will allow placement of the digital video camera at a more suitable location. Blasting achieved minor to moderate damage to the heavy ground support systems installed and major rockmass damage (approximately S2 to S3 damage according to the scale in Table 4).

At the time of writing, planning is underway for the second series of simulated rockbursts, with a view to creating greater damage to the support system.

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Dynamic testing of rock reinforcement using the momentum transfer concept

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ABSTRACT: The West Australian mining industry has an urgent need for the construction of a local dynamic test facility that can perform repeatable dynamic loading on reinforcement systems, support systems and ground control schemes. The West Australian School of Mines, with the assistance of industry and government funding, has developed a test facility using a novel testing process. The facility is in the commissioning phase, and has been proven to enable repeatable dynamic loading of reinforcement systems. Tests have a high level of instrumentation to measure forces and displacements combined with digital video recording. Analysis of these data allow the calculation of energy absorbed from the force displacement curves of the tested system and the impact point in the facility.

1 INTRODUCTION

The purpose of rock support and reinforcement is to maintain excavations safe and open for their intended lifespan. The effectiveness of a chosen ground control scheme impacts the safety of personnel, equipment and the economics of ore extraction. The types of support and reinforcement systems required in a particular application depend on several factors including; strength of the rockmass, geometry of the excavation, stresses present in the rock, blasting practices, weathering and corrosion processes.

Ground conditions are becoming increasingly difficult as the mines in Western Australia are getting deeper (Li et al, 1999). One of the main technical problems faced by underground mines in Western Australia, particularly those that are operating in the Yilgarn Craton, is mining induced seismicity and the related rockbursts. In addition, collaborative university and industry effort in understanding seismic mechanisms and the risk mitigation process is also underway

A need exists to implement measures that protect the work force and mining equipment from rockbursts. This can be achieved by the use of;

- reinforcement and support systems that are capable of surviving rockburst loading,
- exclusion zone and no-entry periods, and

 micro-seismic monitoring systems to improve local understanding of the rockmass response to mining.

1.1 The need for a new dynamic test facility

The design of an appropriate mine sequence and geometry is the primary method to mitigate the effects of mine seismicity in Western Australian mines. The ground control scheme is the main method to mitigate the effects of rockbursts. Consequently, an understanding of the dynamic energy capabilities of reinforcement and support systems, as well as complete ground control schemes to maintain rockbursts requires development.

The Western Australian School of Mines (WASM) has developed a dynamic loading simulation test facility during the last two years. The facility has the ability to test reinforcement systems, support systems, or ground control schemes. The purpose of the simulations at the facility is to answer two main questions;

- How is the released seismic energy absorbed by the ground control scheme or its elements?
- How do the support and reinforcement elements transfer dynamic loads?

The design of the facility required consideration of multiple key components and their interaction. These were;

- Sufficient strength in the engineering design and physical dimensioning of the facility to withstand the impact loads during testing.
- Whether the ground control scheme absorbs the energy or the energy input fails the scheme.
- Determination of energy balance during the test from force displacement response curves. This required the design of instrumentation and monitoring points to provide multiple independent measurement methods of key parameters.
- Knowledge of the energy consumed by elements in the ground control scheme and the test facility impact surface.
- Calculation of the maximum input energy and the relative energy split between the simulated ejected block and the impact surface.
- The impact surface and the reinforcement system will have different relative loading stiffness, and these may influence the other's response to dynamic loading.

1.2 Terminology

The adoption of the following recognized terminology is made;

- Reinforcement System: Comprises the reinforcing element (the bolt), an internal fixture (grout, mechanical or friction coupling), and an external fixture (face restraint).
- Support System: Maybe one or a combination of surface fixtures generally linked to the reinforcement system (w-straps, weld or chain link mesh, shotcrete or fibrecrete, sprayed membrane).
- Ground Control Scheme: Comprises a combination of the reinforcement system and support system.

- Seismic event: A release of built up strain energy from the formation of excavations that does not result in a fall of ground or yielding of a ground control scheme. Energy travels in the rockmass as a wave with frequency and amplitude and is complex in shape.
- Rockburst: Caused by energy waves travelling through the rockmass causing a section of the rockmass to be detached from an excavation boundary that the energy wave encounters. The wave excites both the rockmass that remains behind after the rockburst, and also the rock ejected into an excavation. The ejected rock already has a velocity and does not accelerate further. The actions of the ground control scheme can reduce and stop the displacement of the rock provided it has sufficient capacity. Basically, either a fall of ground occurs or the ground control system yields and maintains the ejected rock.
- Rockmass: The ground surrounding an excavation. During a seismic event, it constitutes both solid ground and fractured ground. Following a rockburst, it constitutes the rock not ejected into the excavation.
- Simulated Rockmass: The drop beam and the steel rings of the test facility prior to impact.
- Ejected Rock: The rock ejected as part of a rockburst that loads or fails the ground control scheme. The ejected rock was a constituent of the rockmass prior to the loading from the seismic event.
- Simulated Ejected Rock: The steel rings integrated with the borehole.

2 WASM MOMENTUM TRANSFER CONCEPT

A literature review has shown that dynamic testing in civil and mining applications using the WASM momentum transfer concept is a novel method. Figure 1 shows three primary components;

- The reinforcement system includes the bolt and surface hardware.
- The simulated ejected rock includes the collar zone, lower pipe length and steel rings.
- The simulated rockmass includes the anchor zone, upper pipe length and the drop beam.

To represent the energy travelling though the rockmass prior to the seismic event becoming a rockburst, an equal velocity of all components is required. This is achieved by dropping all 3 components as one unit.

The WASM configuration uses the proposition that dropping a simulated rockmass, rock reinforcement and support system on to an impact surface, results in a dynamic loading and deformation of the reinforcement system. In order to control the simulated rock mass displacement, one or more components of the ground control scheme must yield. There will be component failure of the system when they are not capable of absorbing the energy.

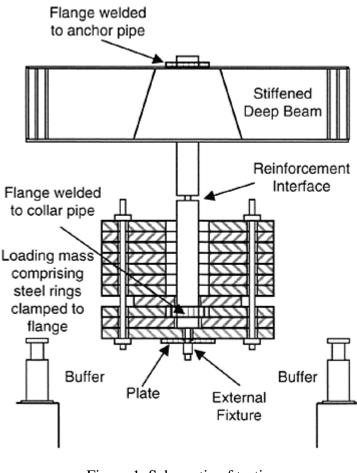


Figure 1. Schematic of testing arrangement showing the major components.

Impact of the drop beam with the impact surface, corresponds to the general rockmass that is not ejected as part of the rockburst coming to rest after the seismic energy travels through the ground. An engineered impact surface is used consisting of hydraulic buffers. The buffers needs to rapidly decelerate the rockmass (anchor zone) and will absorb some of the total energy available at impact.

The simulated ejected rock, represented by the steel rings in Figure 1, will be decelerated according to the properties of the reinforcement system and the buffers. The ability of the reinforcing system to transfer the momentum of the simulated ejected rock to the simulated rockmass and hence the buffers are influenced by the stiffness and strength of the reinforcing system relative to the buffers. The relative stiffness determines the dynamic load on the reinforcement system. The relative velocity of the drop beam

(rockmass) when compared to the ejected rock determines the energy absorbed by the reinforcing system.

In cases were the energy in the ejected rock exceeds the absorption capacity of the system, failure of the elements occurs. Increasing the drop height or increasing the weight of simulated ejected rock raises the energy in the system. The system gains kinetic energy from higher drops.

This mechanism of dynamic testing of the reinforcement system is considered to closely simulate the observed rockbursting behavior in Western Australian underground operations.

3 TEST FACILITY

The intention was to design and construct a dynamic test facility to test reinforcement systems, support systems and complete ground control schemes used in Western Australia. The facility will be utilised by WASM, the mining community, and manufactures of ground control scheme elements. The design and establishment of the dynamic test facility used a logical and engineering approach. This required a number of iterations;

- New idea for dynamic loading ground control systems,
- Develop prototype to test idea,
- Determine energy inputs and scale of tests, whether it was worthwhile building a quarter scale model or go to full scale,
- Conceptual design of the test facility,
- Project proposal and plans to obtain industry and government funding,
- Engineering design of test facility and its components; foundation block, building, guide mechanism, drop beam, buffers, release mechanism,
- Instrumentation requirements to determine the energy balance by monitoring all required forces, accelerations, displacements, strains and capture of the test on digital video,
- Calculation methodology requiring the filtering of instrumentation signals, and the assessment of force displacement curves to calculate energy absorbed by system elements. Thompson et al, 2004 (this conference), explain the calculation methodology in detail.

3.1 Prototype

Figure 2 shows the prototype unit capable of multiple ground control schemes. The unit was composed of a drop box, release point, guides and impact legs.

The prototype box would slide down the guides and the legs would hit an impact surface rapidly stopping the box. The impact brings the box to rest, but the free moving load of gravel (representing the rockmass) dynamically loads the wire mesh and bolts. This occurs because the gravel has momentum.

The upper photos in Figure 2 show the results of a typical prototype test. The mesh supported by four corner bolts has deformed in the middle following impact. The lower photo in Figure 2 is prior to testing and shows an alternative scheme with one centre bolt.

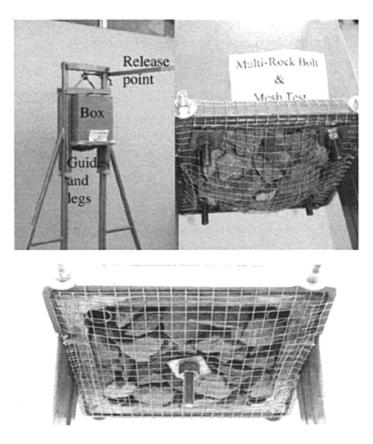


Figure 2. WASM Prototype dynamic loading of ground control scheme.

Regardless of the scale of the test unit, the system must work in a repeatable manner for consistency of energy transfer to the reinforcement and support system.

3.2 Design factors

A number of key design factors required simultaneous consideration, as a modification to one factor would influence another. These in turn would influence the required instrumentation and calculation methodology.

3.2.1 Energy input, size and scale

A decision was made early in the project that all testing should be done on full-scale rock bolts and surface support elements rather than scaling them.

The kinetic energy applied at impact defines the maximum energy available, Equation 1.

$$KineticEnergy_{(total)} = \frac{1}{2}m_{(total)}v_{(impact)}^{2}$$
(1)

The total dropped mass cannot exceed 4500 kg and a maximum velocity of 10 m/s. This equates to 225 kJ.

The weight of the simulated ejected rock loading a reinforcing system is of the order expected in a rockburst event. Yielding reinforcement, such as cone bolts, have a typical installation of a 1.0 m^2 pattern, and depths of ejection are approximately one metre. Hence, a cubic metre is an appropriate base volume for consideration.

Each major component of the system will have load displacement curves, which must be calculated and or measured. The load displacement curves provide the best description of the energy absorbed.

3.2.2 Drop beam size

A consultancy firm specializing in dynamic load calculation through engineering structures was commissioned to specify the drop beam size, reinforcing flanges and webs. The criterion applied to the beam in dynamic loading conditions was 1 mm centre deflection at the maximum load from a reinforcing element. The response of the buffers to the impact load was also included in the modeling of the test system.

3.2.3 Buffers and energy dissipation

While recognizing all dynamic test facilities have energy loss it is not adequate to report the maximum kinetic energy at the impact of a free moving body onto a stationary body. An understanding of the were the energy dissipates is achieved by undertaking an energy balance.

Equation 1 incorrectly assesses the energy that a reinforcing element is subjected to, as it also includes the energy taken out by the buffers. Thinking in terms of kinetic energy absorbed by the reinforcement system, Equation 1 can be modified to Equation 2.

$$KineticEnergy_{(absorbed)} = \frac{1}{2}m_{(cjectedrock)}v_{(relative)}^{2}$$
(2)

The mass of the simulated ejected rock is represented by the steel rings and lower pipe length. The relative velocity is the difference between the simulated ejected rock and the simulated rockmass, as described in Section 3.2.5.

Correct buffer energy dissipation is calculated by the area under the force displacement curve. Buffer displacement is measured either directly from an ultrasonic sensor or indirectly from a double integral of the accelerometer on the drop beam. Load cells on top of the beam measure the force or it can be calculated by a full expansion of force, mass and acceleration relationship. Thompson et al, 2004 fully details this relationship.

Initial specification of the impact velocity on the buffers was 10 m/s. The supplier later adjusted this to 6 m/s. To date, testing buffers at 7 m/s without apparent buffer damage has been successful. Commissioning tests indicate the buffers remove 40% to 50% of the total impact energy.

3.2.4 Integration of 'ejected rock'

It is important that the real life interaction between a rock mass and a borehole is correctly simulated, as detailed in Figure 1. Bolting steel weights together about the load transfer ring welded onto the lower pipe length simulate the rockmass around the borehole.

Bolting of steel rings to the solid base plate on which the lower pipe segment rests integrates the rockmass to the surface of the excavation. The reinforcing element protrudes through the base plate, and is tensioned with the appropriate surface hardware.

3.2.5 Relative velocity and displacement

The maximum drop velocity is not the velocity that the simulated ejected mass will impose on the reinforcing system. The relative velocity occurs between the drop beam slowing down by the buffer action and the 'ejected mass' loading the borehole and the surface restraint.

The relative velocity prior to the impact will be zero, and will be zero again once the buffers have reached maximum compression for a particular energy input, when consumption of all available energy in dynamic loading of the reinforcing system occurs.

3.2.6 Bolt length and support area

The facility is capable of testing any 2.4 m long reinforcing element with a maximum yieldable displacement of 800 mm. Longer reinforcing elements can be tested but the length will depend on the required yield to be assessed.

The testing program of reinforcing elements is to include; 22 mm diameter Cone bolts, 20 mm diameter Gewi bar, 15.2 mm plain strand cable, Garford yielding cable bolt, and 46 mm Splitset. The facility has the capacity to test ground control schemes with maximum surface area of 1.5 m by 1.5 m with four rock bolts on a 1.2 m by 1.2 m pattern.

3.2.7 Bore hole simulation

Appropriate borehole rockmass stiffness can be simulated by the use of thick wall piping, Hyett et al, 1992. Table 1 shows the selected internal steel pipe diameter to equivalent underground applications, and equivalent rockmass stiffness.

3.3 Construction and acquisition of test facility and equipment

The test unit was built on land donated by Kalgoorlie Consolidated Gold Mines. The size of the facility is substantial, as it needs to replicate the energies involved in a rockburst.

Undertaking the construction and commissioning of the dynamic test facility was planned to be in two phases. The first phase required;

1 Design a dynamic test unit and its components, using the WASM momentum transfer concept to provide repeatable tests to simulate a rockburst.

- 2 Construction of the test facility, capable of undertaking tests of reinforcement systems and face restraint.
- 3 Development of instrumentation for the rock reinforcement systems.
- 4 Undertake dynamic testing to simulate rockburst loading on rock reinforcement systems.

A second future phase will modify the test equipment and instrumentation to perform tests on integrated ground control schemes.

The facility had a number of key components involved in the construction and acquisition stage; major foundation block, building for enclosure, guide rails for the drop beam and mass, the drop beam,

Bolt to test	Internal pipe diameter	Wall thickness	Stiffness (MPa/mm)	Equiv. rock (GPa)	
Cone bolt	45 mm	9.3 mm 1389			
Gewi bar	45 mm	7.8 mm	1209	70.2	
Plain cable	76.3 mm	12.8 mm	703	69.2	
Yield cable	76.3 mm	12.8 mm	703	69.2	
Splitset*	82 mm	9.2 mm	472	49.9	
Rock	45 mm	Infinite	1174	65	

Table 1. Effective pipe stiffness.

* This is does not included the grout to simulate the rockmass. The grout has a 45 mm hole, into which the Splitset is Jumbo driven.

buffers/impact surface, and the release mechanism. The complete facility is shown in Figure 3, but will be detailed in the figures that follow.

3.3.1 Foundations and building

The design of the major foundation block was to withstand a direct impact at maximum velocity and mass without failure or damage to the foundations. Realistically, repeated direct impact on the foundations would deteriorate the foundation block. The foundation block is heavily reinforced with steel bars and panel fibres (Figure 4). The foundation block has an isolation layer from the remainder of the facility to minimise vibration loading of the test building.

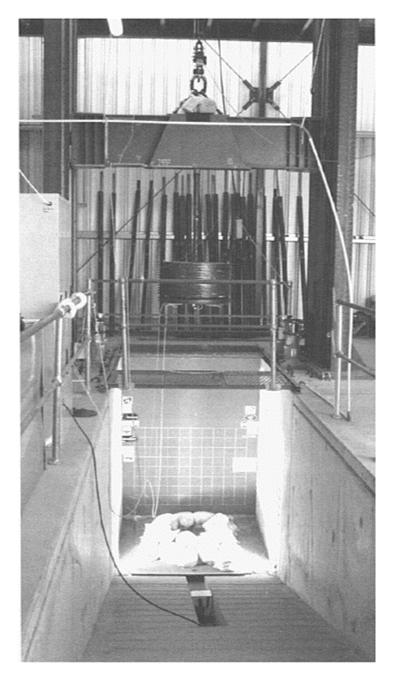


Figure 3. Completed WASM test facility.

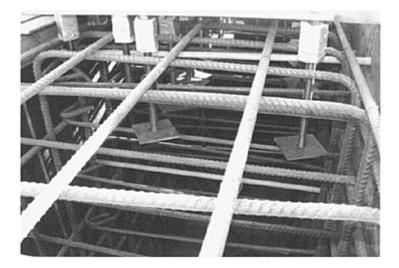


Figure 4. Detail of the foundation block, with tie down bolts for buffers and guide rails.

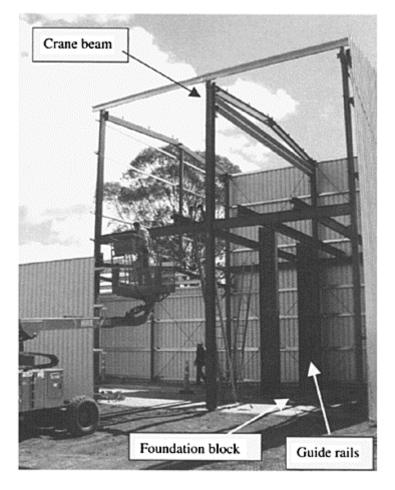


Figure 5. Building construction.

The test building is fifteen metres long by eight metres wide. The rear section of the building is 10 m high and houses the drop crane and test pit (Figure 5).

3.3.2 Guide rails

The guide rails for the drop beam are six metres high. This allows a maximum impact velocity of 10 m/s. This height also takes into account the height of the buffers and depth of the beam. The guide rails were designed to withstand any potential failure of the guide system at impact of the beam onto the buffers.

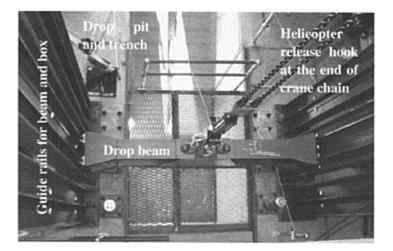


Figure 6. Guide rails for drop beam and release hook.

The I-beam guide rails use a slider and shoe system similar to those used for lifts in multistory buildings. The guide rails, direct the beam during the free fall and control the beam at impact (Figure 6).

3.3.3 Drop beam

The drop beam was constructed from a 610UB101 steel section. Additional webbing was fitted at the impact point, plus, side, top and bottom panels in the middle of the beam to control bending from bolt loading. This strengthened the centre cut out for the simulated borehole. A thick wall pipe was welded into the middle of the beam in order to accommodate the simulated borehole. End plates are welded onto the beam to attach the guide shoes. Figures 3 and 6 both show the beam.

3.3.4 Release mechanism

A helicopter release hook was chosen for the release mechanism, for the following reasons; the release hook load carrying capacity, inherent safety in the aeronautical design, and ease of working with the beam and mass, and the ability to release the mass from a remote safe distance. The hook was combined with a shock absorber in order to substantially reduce any dynamic loading of the crane beam and building (Figure 6).

3.3.5 Impact buffers

The choice of impact surface required the selection of an engineered and repeatable rapid deceleration of the drop beam at impact. The Oleo buffers provide the response. The direct impact onto the foundation block or into a sandbox does not provide this reponse.

The principal of the Oleo buffer is driving an orifice over a metered pin in an oil filled well (Figure 7). Dissipation of the energy occurs through turbulent flow and heating of the oil. Normal applications of these buffers are rail rolling stock and aircraft landing gear.

There is an expectation that changing the impact velocity, 'ejected rock' mass, and reinforcement system will load the buffers in subtlety different ways. The relative stiffness and strength of the buffers compared

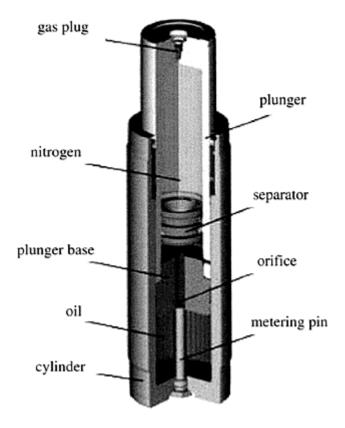


Figure 7. Section of Oleo buffer, from http://www.oleo.co.uk/.

to the reinforcement element is an influential process in the dynamic load transfer to the reinforcement system. It is possible to alter the 'stiffness' of the buffer response by having a mass on top of the buffer. This occurs because the effective plunger mass will be larger and require a higher momentum to commence movement. It is also possible to use alternative impact systems.

3.4 Instrumentation

In order to undertake fundamental analysis of the mechanisms of load transfer, it was identified that test facility instrumentation was required to measure force, displacement, acceleration and strain of the;

- Reinforcement system (bolts, surface hardware, collar and anchor),
- Simulated ejected rock (the integrated steel rings and lower pipe length),
- Simulated rockmass (the drop beam and upper pipe length), and
- Buffers (the impact surface).

Instrumentation was designed or selected to record force, displacement, acceleration and strain in small time increments. This allows solution of force displacement curves and the relative velocity between the 'ejected rock' and the 'rockmass'.

The design and selection of instrumentation and data acquisition system involved a number of key requirements;

- Integration of digital video with sensor data,
- Relatively high speed digital video capture,
- High speed acquisition of data per sensor channel,
- A large number of channels,
- Support strain gauge, integrated circuit protocol (ICP), and direct voltage output instrumentation,
- Software control of sensor data acquisition and digital video capture,

3.4.1 Data acquisition

A National Instruments PCI6071E data acquisition (DAQ) board controls the acquisition of data from all sensors. The card is configured for 32 differential input channels utilizing 12 bit sampling. With the current sensor requirements, the facility only requires 21 channels.

The DAQ channel sample rate is 25,000 samples per second per channel. Sampling occurs simultaneously on all channels. The sample rate determines the smallest time interval recorded, but the highest recordable frequency for cyclic analogue signals is half the sample rate, Nyquist Theorem.

The underlying data acquisition software is from National Instruments, but the video control software is Midas from Xcitex. Midas software provides a frontend operating system for recording the tests.

All data can be output to an excel spreadsheet, displaying the sensor data along with point analysis of object locations from auto-tracking the video. A video file is also generated. Sensor data and video information are both time-coded and interlinked allowing combined assessment.

The DAQ system has a two-second window for data and video with a trigger option for acquiring the data. Sensors and video are continuously acquiring information once powered, but requires a trigger signal to start the recording process. Instrumentation gathers two seconds of data during a test but analysis requires less than 0.6 seconds of data. The division of instrumentation channels provides for 8 strain gauge channels (filtered by a National Instruments SC2043SG board), and 12 ICP configured channels via a PCB Piezotronics 483A signal conditioner with BNC connections. The remaining 12 channels are DC voltage channels with BNC connections to the DAQ board. Figure 8 shows a schematic of the DAQ boards, computer and instrumentation.

3.4.2 Sensors

The selection of a combination of permanent and temporary sensors, and the possibility for sensor damage requires a secondary means of acquiring or calculating crucial data. Thompson et al, 2004 shows the load transfer diagram for all components and their interactions in a test.

All commercial equipment was supplied with calibration factors, and calibration of purpose built equipment was undertaken as part of the project. The quality

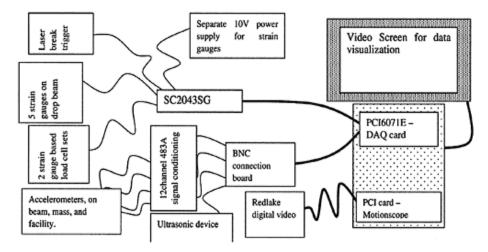


Figure 8. Schematic of instrumentation and data acquisition.

of the measured data point is a function of the combined accuracy and precision of both the sensor and the DAQ board.

3.4.2.1 Accelerometers

All accelerometers are from PCB Piezotronics. Three 356A02 triaxial 500 g units were selected. The units have an acquisition range of 1 Hz to 5 kHz, to which a mechanical filter has been fitted to protect the unit from high frequency and sensor saturation due to metal to metal contact. This reduces the upper range to approximately 2 kHz. The sensors and board have a combined accuracy of 2.35 m/s^2 in their current configuration.

A uniaxial shock accelerometer was also selected (the 350A13). The unit is electronically filtered and generates much cleaner signals. The sensor and board

configuration has a range of 1 Hz to 10 kHz (± 1 dB) with a combined error of 46.7 m/s², but it should be possible to improve this to 9.0 m/s².

Accelerometers are placed in key locations including;

- On top of the drop beam above the buffer,
- Underneath the beam two thirds of the distance between the impact point with the buffer and the centre hole,
- On the simulated rockmass (Figure 9). Both the 10,000 g shock accelerometer and a 500 g accelerometer are located here for side by side testing to understand the difference in sensor response.

3.4.2.2 Load cells

Collar force and the anchor force are recorded by purpose built 300 kN load cells with spherical seats. A single load cell is used to measure the collar force (Figure 10) with a combined sensor and board error

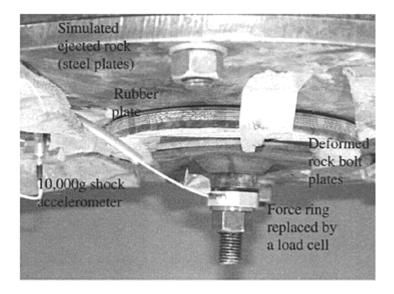


Figure 9. Shock accelerometer and surface hardware.

of 1.28 kN (and 5.3 kN during commissioning tests), this replaces a force ring (Figure 9) which proved to be inappropriate for the application.

Four load cells, wired in a full bridge configuration were used to measure the anchor force, Figure 11. The load cells are located between the backing plate for the simulated borehole and the top of the beam. The simulated borehole is bolted to the beam through the load cells. The load cells and DAQ board have a combined error of 3.53 kN (and 14.1 kN during commissioning tests).

3.4.2.3 Ultrasonic motion sensor

The ultrasonic motion sensor was selected in order to assess compression of the buffers during deceleration of the beam. The selection of the ultrasonic device

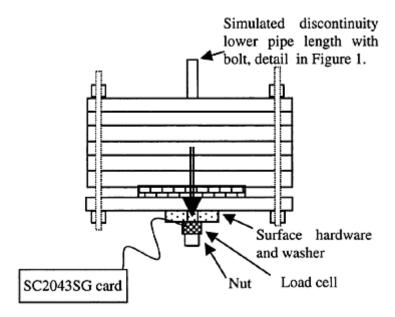


Figure 10. Load cell at the collar measuring force.

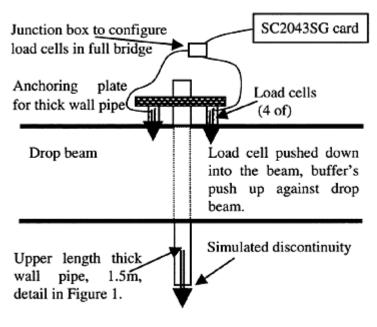


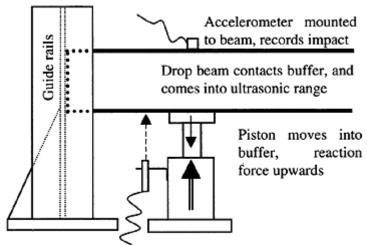
Figure 11. Load cell set measuring anchor force.

was dependent on meeting the following criteria;

- No direct contact to beam, hence no damage to sensor,
- Ability to measure the maximum buffer compression of 104 mm,
- A DCvolt output compatible with the DAQ board range.

A HydePark SM606A02 was selected for this purpose. However, the selected unit has a comparatively slow sample rate of 1.5 milliseconds (ms). The digital over sampling technique used by the DAQ card provides a step function record of the buffer compression. This requires filtering as the buffer is a smooth travelling device.

The ultrasonic unit has an accuracy of 0.69 mm, with a board variation of 0.03 mm. The sensor was mounted in a bracket on the side of the buffer as shown in



Ultrasonic probe records beam movement as the buffer is compressed and recovers, direct voltage output to DAQ.

Figure 12. Ultrasonic measuring buffer compression and accelerometer on beam above buffer.

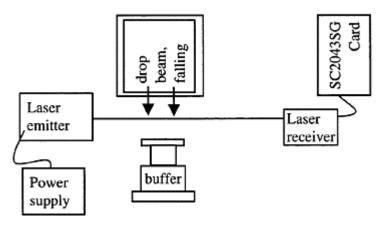


Figure 13. Laser break trigger of instrumentation.

Figure 12. The accelerometer mounted on top of the beam above the oleo is used for determining impact with the buffer. This is due to inaccuracy of 1.0 mm to 1.5 mm in beam displacement from using the ultrasonic device, as the beam is moving into the far limit of its detection range. In the future, other alternatives will be examined in order to provide accuracy to 0.1 ms.

3.4.2.4 Laser break and triggering

The drop beam moves through a laser beam starting the recording process. Breaking the laser beam sends a 5-volt pulse to the SC2043SG board (Figure 13). The data window closes 2 seconds after the trigger signal and the trigger allowance. The trigger allowance can be set from plus 2 seconds to minus 2 seconds.

3.4.2.5 Physical measuring

Physical measurements are taken before and after each test in order to confirm sensor measurement and provide an understanding of yield of the reinforcement system. Required measurements include;

- Toe of bolt displacement (Figure 14),
- Separation displacement at the discontinuity (Figure 15),

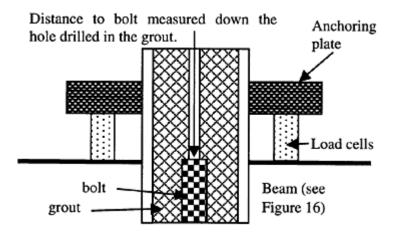


Figure 14. Measuring anchor of bolt displacement.

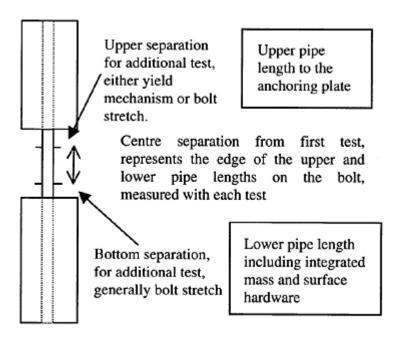


Figure 15. Measurement of separation at the simulated discontinuity at each test.

- Torque on nut, only undertaken during tensioning of the bolt prior to the first test,
- Surface plate deformation.

3.4.2.6 Strain gauge

Strain gauges attached to the drop beam are used to determine compressive and tensile strains in the beam from the dynamic loads (Figure 16). Micro-Measurement EA-06–500BL–350 strain gauges were selected for this application. To improve the sensor sensitivity of the strain gauges and load cells, additional excitation voltage was required. Excitation voltage was recorded at each sample point and were used for strain gauge based calculations.

The results of the tests (as shown in Table 2) indicate that the beam behaved stiffly, as none of the input energy was lost through beam deflection. The straingauge board has a resolution of approximately 24 microvolts which approximates to 5 microstrains.

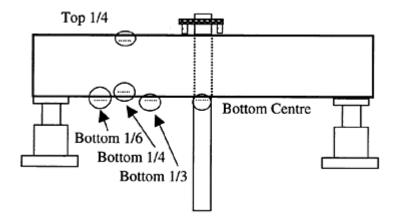


Figure 16. Strain gauge locations on drop beam.

3.4.3 Camera recording

Digital video capture of the drop occurs at a rate of 250 frames per second. The digital video camera has a pixel resolution corresponding to 3.3 mm of the viewing test area. Time coding of the video allows interlinking with sensor data. Higher sample rates are possible, however there is a loss of frame area.

The auto-tracking software can calculate displacements, accurate to approximately half of one pixel, 1.7 mm. When velocities approach 1/2 pixcel/frame rate comparatively large and unrealistic steps occur in the data. Therefore, in order to obtain a linear displacement versus time curve, the video captured displacement data requires smoothing. With knowledge of the acceleration and mass of objects components, it is possible to calculate force.

The camera was mounted at 15° above the horizontal, which allows viewing of the plate and surface hardware during each test. A geometric correction factor was developed to account for the camera mounting angle. However, the analysis can only use points on the centre line of the drop. Figure 3 shows the camera at the bottom of the trench, with the grid on the back wall of the pit used for scaling.

4 TESTS

During the development and commissioning phase of the facility, 11 bolts were dynamically tested using 63 drops. A summary of the tests carried out in establishing the facility, developing the instrumentation protocols and the analysis methodology are shown in Table 2. Bolts tested included;

- Cone bolts (22 mm diameter bar),
- Gewi bars (galvanized 20 mm diameter), and
- Smooth bolts (cone bolt with the cone cut off).

Ejected rock masses trialled;

- 500 kg (too light, drop beam is 645 kg),
- 1500 kg (an improvement), and
- 2000 kg (a good starting test weight).

Table 2. Summary of commissioning test program.

There are a summary of commencements of the programme					
Bolt #	Туре	Drops	Heights	Summary of results	
14	Smooth	9	100–400	287 mm slip, concrete mass 500 kg, cut out welds from simulated discontinuity and start noise isolation work	
13	Smooth	3	400= 3 m/s	125 mm slip, concrete mass, instrumentation learning process, noise isolation	
13	Smooth	1	1000	325 mm slip, steel rings replace concrete mass, instrumentation improvements	
1	Gewi	3	1000	Stripped nut (was fully engaged), no load transfer ring on the outside of the borehole pipe	
2	Gewi	21	1000– 1850	Load transfer ring standardized for fully bonded bolts, 500 kg steel mass, additional and change sensor locations, 190 mm stretching/slip, consistent bolt changes with each drop, substantial work on strain gauge setup	
31	Cone	10	1800= 6 m/s	500 kg mass, load transfer ring, 104 mm total slip and yield, buffer response assessed, consistent bolt change with each drop	
34	Cone	10	1800	500 kg mass no load transfer ring, 108 mm of total separation. 20 ms loading time on the anchor force, increase strain gauge excitation voltage, consistent bolt changes with each drop	
34	Cone	3	1800	500 kg mass, change force ring for load cell at the collar. Similar forces recorded by collar and anchor load cells, and same time of peak occurrence, due to debonding and yield of the cone bolt	
34	Cone	3	1800	500 kg mass, buffer compression device trialed to increase buffer stiffness, no gain in shock, but reduction in travel distance of buffer	
4	Gewi	1	1280= 5 m/s	1500 kg mass, approximately 19 kJ of total energy input, stripped nut (was not fully engaged) and pulled the lower pipe length off the grout around the bolt. Anchor and collar forces suggest 80 kN required to cause borehole slip. All pipes to have additional friction by the use of shear pins installed through the wall of the pipe and into the grout. Buffer compression trail continued	
38	Cone	3	1280	19 kJ input per test, estimate 10 kJ absorbed by the bolt yield per test. The shock recorded by the accelerometer on the mass is lower with the increase in mass. The yielding system can only apply force at a fixed maximum amount so it takes longer to	

				slow the ejected rock. Loading time 70 ms, 1 14 mm of total
				separation from 3 tests. Buffer compression trial continued
37	Cone	1	1275= 5 m/s	1500 kg mass, 50 ms of load at anchors at 180 kN to 220 kN, 48 mm of separation. Approx 8.5 to 10.5 kJ absorbed by bolt, rest into buffer of 19 kJ input
37	Cone	3	1835= 6 m/s	Continuation of testing, yield of the bolt, anchor force load curve is stiffening with each test, possibly due to work hardening of the bolt, 197 mm of yield from the 3 tests
37	Cone	4	2500= 7 m/s	Continuation of testing, yield of the bolt, anchor force high for short time periods 50 ms, on the 6th test there is cone slide with a duration of 130 ms. 380 mm of separation from the 5th to 7th drops, and then bolt fails on the 8th drop. End of trials with buffer compression
42	Cone	4	1265– 1845	1500 kg mass. First drop at 5 m/s rest at 6 m/s. Load cells recording on collar and anchor, 155 mm of total separation from the first three test, then stripped nut on the 4th test (was not fully engaged) Cone movement of 40 mm and bolt stretch of 115 mm recorded
11	Gewi	3	1245– 1845	2000 kg mass, first test uneven and slower then ideal, 117 mm of separation by the second test, gewi bar pulled out of grout on the third tests. Tested gewi's are galvanized and have a rope thread on less than half the diameter of the bolt

Impact velocities trialled;

- -3 m/s, (this is the minimum velocity for the facility),
- -5 m/s, (this is the preferred starting impact velocity),
- 6 m/s (buffer's re-rated impact velocity), and
- 7 m/s.

A subtsantial amount of work as required during the development and commissioning. The issues addressed were;

- Testing procedure and sequencing,
- Alignment of the guide rails for smooth drops,
- Quality requirements in test sample preparation,
- Integration of the simulated ejected rock to the borehole, to the grout to the reinforcing element,
- Response of the buffers to dynamic loading and how to adjust that response,
- Stopping metal to metal contact to prevent instrumentation saturation,
- Instrumentation and DAQ configuration to obtain optimal results,
- Filtering and analysis techniques. This process is discussed by Thompson et al, 2004.

4.1 Example results

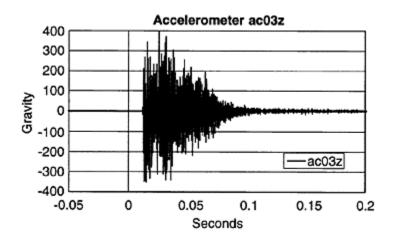
Table 2 shows total displacements achieved from a series of dynamic tests conducted on various reinforcement systems. Tests conducted as part of the commissioning of the facility assisted in determining the conditions for sensor saturation and data loss.

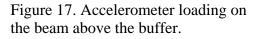
Example results are shown in Figures 17–19. These are raw results prior to filtering. Thompson et al, 2004, presents analysis with the filters, the results and examines the force displacement curves for the system.

Figure 17 shows the response of a 500 g accelerometer, without a mechanical filter, mounted on top of the beam above the buffer. The accelerometer was positioned to measure acceleration of beam prior to impact with the buffer. It also measures beam deceleration and is used to interpret the acceleration of the buffer after beam impact. This is a clear example of filtering the results to obtain the true signal. Prior to impact the beam is descending, and the sensor records a signal of 1 g.

Figure 18 shows the response of the collar and anchor load cells. The results are for the second drop on a cone bolt. A loss of collar tension due to steel yielding of the bar was experienced during the first test. During the beam descent, the collar load cell recorded the weight of the simulated ejected rock, approximately 20 kN. The load cells provide the best indication of the duration of load on the bolt. Load duration in this example is approximately 70 ms, with a rise time of 5 ms. The difference between the collar and anchor force is due to measurements of different masses (ejected rock versus ejected rock and upper pipe length) and deceleration mechanisms (reinforcement element versus buffer's).

Figure 19 shows the displacement of the buffer and the shock accelerometer on the simulated ejected rock. The results indicate that the ultrasonic record of buffer displacement requires filtering to smooth the data record, and the difference in timing of beam impact onto the buffer as determined by the ultrasonic sensor and accelerometer needs to be improved to 0.1 ms. The shock accelerometer has a greater combined error than the 500 g accelerometers however, it has inbuilt filtering, hence less additional filtering are required to assess the true response.





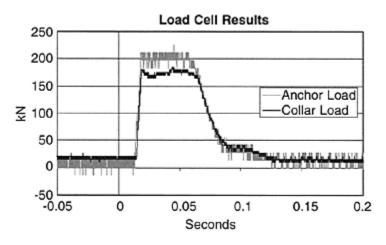


Figure 18. Load cell response to dynamic load.

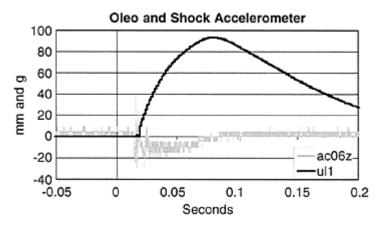


Figure 19. Buffer displacement and shock accelerometer.

4.2 How momentum transfer relates to real conditions

It is considered that the WASM momentum transfer concept used by the test facility accurately simulates ejected rockmass during a rockburst. This is mainly due to the nature of energy transmission through a rockmass. Some of the seismic source parameters that are generally accepted to describe the complexity of energy traveling as a seismic wave include; Moment Energy, Radiated Energy, corner frequency, and radius of slip.

Energy waves originate from a seismic source and travel through the rock mass in most cases. The seismic source is generally located at a distance greater than the fractured stress relieved zone around an excavation. For a strain burst on the surface of an excavation to occur, the rockmass must have had no stress fracturing prior to the event, in order to allow the burst to occur on the surface of the excavation. As the waves approach the excavation, the stable rockmass, the reinforcement elements (rock bolts), and the unstable fractured ground (potentially the ejected rock) are being 'excited' by the energy in the waves.

A process of ejection of the existing fractured ground, or freshly fractured ground, occurs due to the interaction of the energy waves and the stress field about an excavation. This ejection process is not considered instantaneous, yet it is also not comprehensively understood. However, the ejection process is considered to occur very rapidly, and is a result of energy waves overlapping and interacting with the existing solid or fractured rockmass around an excavation.

At the WASM facility, the energy traveling though the rockmass, prior to ejection, is simulated by dropping all the components together. The test method simplified the excitation wave. The buffer provides the loading phase (ramp up) of energy transmission into the ejected rock. Impact rapidly applies load to the reinforcement element, sometimes to the maximum capacity of the element at which point it will yield or fail. Initial testing shows that the ramp up time from initial loading to maximum load is less than 5 ms.

5 CONCLUSIONS

The project has successfully advanced since inception of the facility. The design, construction and commissioning of the facility was undertaken within a two year period. The facility has the capacity to represent rockbursts by a new loading methodology which can be applied to the ground control scheme elements.

It is proven from the commissioning tests that the WASM dynamic test facility can provide significant loading of the reinforcement system at the accepted static capacity of reinforcement system elements. These loads are applied for short durations of time with a very rapid rise time (less than 5 ms) representing the shock load to the reinforcement system.

The outcome of the project will be to determine the energy absorption capacity of the reinforcement and ground support systems and different fully integrated ground control schemes.

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Simulation and analysis of dynamically loaded reinforcement systems

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ABSTRACT: The performance of reinforcement systems when subjected to dynamic loading may be substantially different to the performance measured in quasi-static tests. In order to investigate the extent of the differences in behaviour, a new testing facility has been developed to apply dynamic loads to reinforcement system specimens set up in double embedment configurations. A computer simulation of the test facility has been developed to facilitate test data processing and interpretation. The tests are monitored electronically and by high speed digital video. The results from the various forms of instrumentation are filtered and then combined and analysed. The analyses enable the energy absorption characteristics of different reinforcement systems to be characterised. An example is presented of a simulation and test on a reinforcement system. The ability to apply multiple impacts to simulate repetitive loadings from seismic events is one major asset of the testing facility.

1 INTRODUCTION

A new test facility for dynamic testing of reinforcement and support systems has been constructed in Kalgoorlie by the Curtin University of Technology, Western Australian School of Mines. A comprehensive description of this facility is given by Player et al. (2004).

Clearly, the analysis and interpretation of the results are important components of the testing procedure. Two computer programs have been developed. The first one is used to simulate the behaviour of different types of reinforcement systems when subjected to the types of loading in the test facility and the second is used to analyse the results obtained.

The simulation software was developed to aid in understanding the interactions between the various components of the test facility, to design the instrumentation required to quantify the performance of reinforcement systems and to assist with interpretation of results. The analysis software has been developed to filter the time varying data collected during testing and to perform calculations of the reinforcement force-displacement response and energy absorption.

The two computer programs are described in detail and examples of their use are presented.

2 DESCRIPTION OF TESTING FACILITY

Brief descriptions of the new test method and associated instrumentation and monitoring system as it relates to the computer simulation of the tests and the analysis of the testing data follow.

2.1 Components

Figure 1 shows a schematic of the major components of the new testing facility. The three components are:

- The Reinforcement System.
- The Collar Zone.
- The Anchor Zone.

In the field, the latter two components correspond to a detached block of rock and stable rock, respectively. The test facility attempts to simulate the loading on the reinforcement within and between these two zones.

The following three sections describe each of these components in more detail.

2.1.1 Reinforcement system

All reinforcement systems are contained within two abutting steel pipes. The lower, collar pipe simulates the collar zone of the reinforcement system and the upper, anchor pipe simulates the anchor zone.

2.1.2 Collar zone

The collar zone consists of the collar pipe and a welded steel flange to which the loading mass (comprising a number of separate steel plates) is clamped. The reinforcement system plate is clamped between the loading mass and the external fixture.

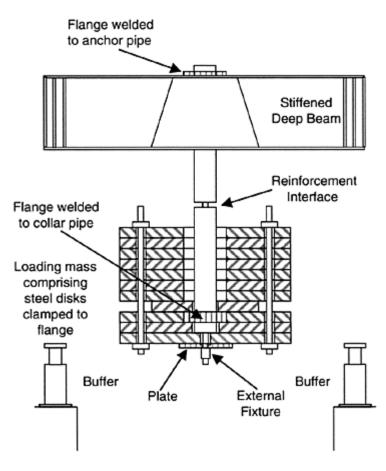


Figure 1. Schematic of new testing facility showing the major components and their arrangement.

2.1.3 Anchor zone

The anchor zone comprises a deep, stiffened steel beam to which the anchor pipe is connected. The reinforcement system transfers load from the collar pipe to the anchor pipe.

The anchor zone behaviour is directly affected by the beam impact surface. Initially, commercially available hydraulic buffers were selected to protect the concrete foundations during commissioning of the test facility. A number of methods are available to modify the behaviour of these buffers when greater experience has been obtained in using the new testing technique and facility. It is also possible to replace the buffers with other devices that have different responses to impact.

2.2 Reinforcement load transfer mechanisms

In order to design the testing facility instrumentation and to simulate reinforcement testing, it was necessary to define the component interactions, load transfer mechanisms and forces shown in Figure 2.

The symbols used in this figure represent:

- $M_A \qquad \qquad mass \ of \ beam.$
- $M_P \qquad \qquad mass \ of \ anchor \ pipe.$
- M_C loading mass (including collar pipe).
- M_B mass of buffer piston.
- F_{RA} element force at a discrete internal fixture.

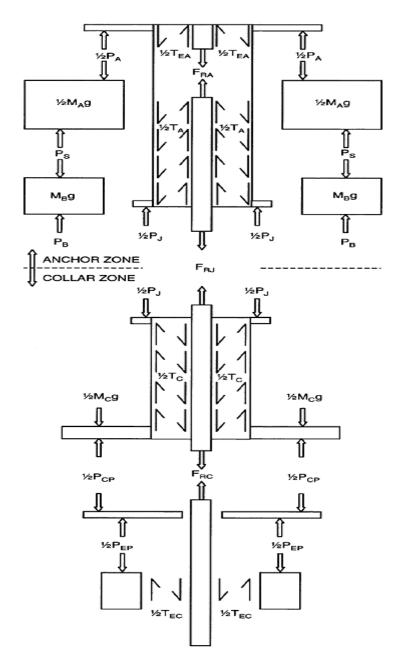


Figure 2. Schematic of load transfer mechanisms for a reinforcement system in the WASM Dynamic Test Facility.

- F_{RJ} element force at the interface between anchor and collar zones.
- F_{RC} element force at the collar fixture.
- T_{EA} load transfer between element and wall of pipe (or borehole) at a discrete anchor.
- T_A load transfer between element and wall of pipe (or borehole) in anchor zone.
- T_C load transfer between element and wall of pipe (or borehole) in collar zone.
- T_{EC} load transfer between element and fixture.
- P_J force transfer at the interface between the anchor and collar zones.
- P_{CP} force transfer between the collar zone and plate.
- P_{EP} force transfer between the fixture and the plate at the collar.
- P_A force transfer between anchor pipe and beam.
- P_S force transfer between the beam and the buffer.
- P_B internal buffer force.

Reinforcement system displacements and deformations that occur during testing are an important aspect of the analysis. The global displacements of the components and the internal reinforcement system displacements are shown schematically in Figure 3.

In this figure, the symbols represent:

- $u_A displacement of the beam (after impact assumed to be also the displacements of the upper pipe (u_P) and the buffers (u_B)).$
- u_c displacement of collar pipe.
- u_{PL} displacement of plate.
- u_{RE} displacement of external fixture
- W_{RA} displacement of reinforcement anchor/free end relative to anchor pipe.
- W_{RJ} reinforcement displacement across interface.
- W_{RJA} displacement of reinforcement relative to anchor pipe at interface.
- W_{RJC} displacement of reinforcement relative to collar pipe at interface.
- W_{RC} displacement of reinforcement relative to collar pipe at collar.
- W_{RE} displacement of reinforcement relative to external fixture.
- W_{EP} displacement of fixture relative to plate.
- w_{PC} displacement of plate relative to collar pipe.

Note that for certain types of reinforcement systems, some of the load transfer mechanisms and displacement will not be relevant. For example, if a system does not have a discrete anchor, then $F_{RA}=0$ and the load transfer in the anchor zone is T_A . On the other hand, for a bolt anchored by an expansion shell alone, $T_A=T_C=0$. In other cases, there may be no plate and external fixture at the collar.

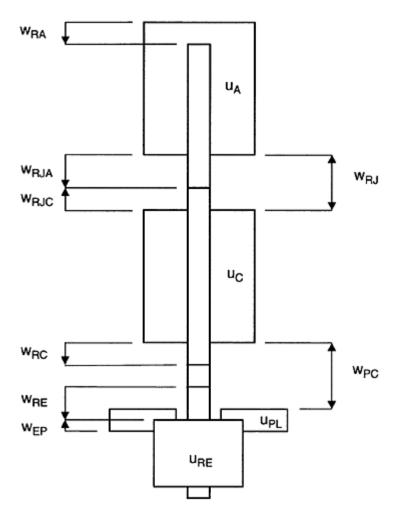


Figure 3. Displacements and deformations of components and reinforcement system in the WASM Dynamic Test Facility.

The symbols shown in Figure 2 and Figure 3 and defined in this section are all used in the formulation of the simulation and analysis of reinforcement system response to dynamic loading. Some of the symbols are used in the captions to figures to indicate where measurements of displacements, accelerations and forces are made.

2.3 Test procedure

A test involves dropping the beam, reinforcement system and loading mass from a known height to impact on the buffers. After the initial impact, the combined beam and buffers comprise the anchor zone.

2.4 Measurements

In order to quantify the behaviour of the complete testing facility, and in particular the reinforcement response, measurements are made at various locations on the components of the testing facility.

2.4.1 Forces

The forces at the collar (between the plate and the external fixture of the reinforcement system) and in the anchor zone (between the anchor pipe and the beam) are measured by electronic (strain-gauged) load cells (not shown in Figure 1). These cells measure P_A and P_{EP} , respectively.

2.4.2 Displacements

Displacements of the anchor zone and the collar zone are monitored. The beam/buffer displacement (u_A) is measured by a motion sensor and the displacement of the loading mass (u_C) and the external fixture (u_{RE}) are derived from post-processing of a high speed digital video camera recording.

At the completion of a test, displacements W_{RA} , W_{RJ} , W_{RJA} and W_{RJC} are measured manually using a vernier.

2.4.3 Accelerations

Accelerations of the anchor zone (beam— \ddot{u}_A) and the collar zone (both loading mass— \ddot{u}_A and the plate— \ddot{u}_{PL}) are monitored by accelerometers. The results from these accelerometers are also used to estimate velocities and displacements of these components.

2.4.4 Strains

Strains in the beam are monitored by strain gauges placed on the tensile and compressive faces.

2.5 Data recording

Data in the tests are recorded electronically and stored by a high speed data logger and visually by a high speed digital video camera. The data are synchronised by software.

2.5.1 Computer data acquisition system

Data acquisition occurs at 25000 Hz (i.e. readings are taken and stored every 0.04 msec). The details are given by Player et al. (2004).

2.5.2 High speed digital video camera

Visual recording of the tests occurs at 250 Hz (i.e. 1 frame every 4 msec). Again, the details are given by Player et al. (2004).

2.5.3 Software

The instruments and data collection are configured using Redlake *MotionScope®* software (Redlake Imaging Corporation 1999). This software also included data visualisation, editing and storage facilities.

3 SIMULATION OF TESTING FACILITY

In order to aid in understanding the interactions within the new test facility, computer based simulations of the complex interactions between the components were attempted.

The interactions were simulated by assuming characteristic responses for the various components and analysing them using Newton's second law (i.e. Force=Mass×Acceleration).

3.1 Description of test procedure

In order to simulate the test facility, it is first necessary to qualitatively describe the mechanisms associated with various different phases in the test and to then attempt to model these using well-established computational models for the particular mechanism. The phases of the test can be summarised as follows:

- 1. The beam, reinforcement and loading mass assembly are lifted to a known height above the reference surface (uncompressed buffer piston).
- 2. The complete assembly is dropped.
- 3. The beam impacts on the buffer piston (the behaviour instantaneously is governed by the impact equation. That is, Impulse=Change in Momentum which mathematically is:

$$|Fdt=m\Delta v$$
 (1)

It is assumed during impact that the mass is the beam and upper pipe.

4. After impact, the beam will be moving more slowly and the buffer piston will be moving. Momentum will be conserved. However, there will be energy lost in the impact and the new velocities of the beam and buffer piston will depend on the notional coefficient of restitution (e) for the contacting surfaces. The coefficient of restitution is defined by:

$$\mathbf{e} = -\frac{\left(\mathbf{V}_{At} - \mathbf{V}_{Bt}\right)}{\left(\mathbf{V}_{A0} - \mathbf{V}_{B0}\right)} \tag{2}$$

where V_A is the velocity of the beam, V_B is the velocity of the buffer piston, and, the subscripts 0 and t are these velocities before and immediately after impact, respectively.

This equation, combined with the conservation of momentum equation:

$$m_A V_{A0} = m_A V_{At} + m_B V_{Bt}$$
(3)

enables the calculation of the velocities of the beam and buffer piston immediately after impact. It is possible that immediately after the initial impact, the piston will for a very short time move more quickly than the beam until the internal resistance slows the piston.

- 5. After the internal resistance slows the piston, it is assumed that the beam and buffer mass are in contact. The behaviour at the contact between the beam and buffer mass is controlled by the response of the thin, stiff rubber pad. This pad is used to eliminate metal to metal contact and to minimise the noise sensed by the accelerometers during and following the impact.
- 6. As the beam slows, the relative velocity between the collar zone (being loaded by the mass) and anchor zone (being restrained by the beam and buffers) will increase from zero.
- 7. The relative velocity will result in relative displacement across the interface between the collar zone and the anchor zone and force will develop in the reinforcement system.
- 8. The force in the reinforcement system will now attempt to retard the loading mass and to accelerate the beam.

The acceleration of the beam will be resisted by the buffers and the inertia of the beam.

3.2 Components

As indicated previously, the dynamic testing facility can be assumed to consist of three major components:

- The Reinforcement System.
- The Collar Zone.
- The Anchor Zone.

For the purposes of analysis, the anchor zone is separated into the beam and the buffers.

3.2.1 Reinforcement system

For evaluation purposes, the reinforcement system response is defined by the force displacement at the interface between the anchor and collar zones. The segmental non-linear ("elasto-plastic") response for a rock bolt with a yielding anchor is shown in Figure 4. This response curve is based on the results obtained for the test to be described in Section 4.

3.2.2 Loading mass

The loading mass is assumed to be rigid and is coupled to the lower collar zone pipe. The loading mass also interacts with the external fixture of the reinforcement system.

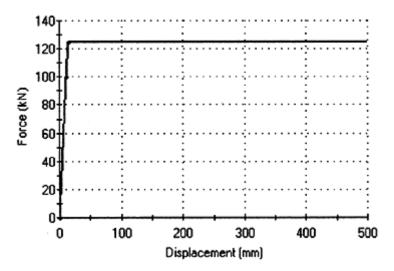


Figure 4. Force-displacement response of a yielding reinforcement system.

3.2.3 Beam

The beam is assumed to be rigid and coupled to the upper anchor zone pipe of the reinforcement system.

3.2.4 Buffers

The buffers are designed to dissipate energy by high speed flow of hydraulic fluid through an orifice. For the particular type of buffer chosen, the orifice area decreases as the buffer compresses. This results in higher velocity of fluid flow and higher energy losses as the buffer compresses. The actual variation of orifice area and piston displacement and an approximate relationship (ignoring fluid compressibility) between pressure and velocity were obtained from the buffer supplier. The static (i.e. velocity=0) response of the buffer is shown in Figure 5 while the response to an impact of 1000 kg at 10 m/s (50 kJ) is given in Figure 6.

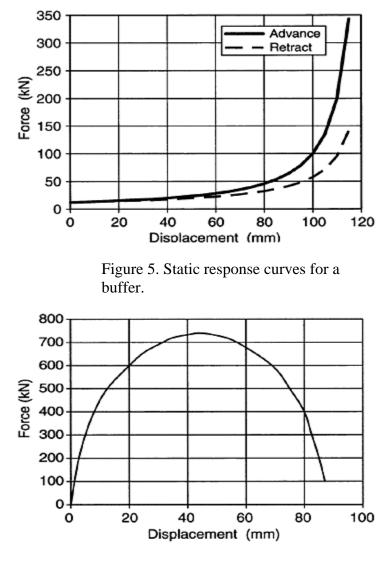


Figure 6. Theoretical forcedisplacement response of a buffer subjected to an impact energy of 50 kJ from a mass of 1 tonne.

3.2.5 Impact surface response

The impact surface between the buffer piston and the beam comprises a hard rubber pad, approximately 10 mm thick. At this stage, it is assumed that the pad compresses fully during initial impact and the total displacement is negligible compared with other component displacements.

3.3 Component interactions

The forces acting on each component shown in the free body diagram (Figure 2) can be used in a time stepping analysis method to predict the behaviour of all the components during a simulated test. The equations of motion at each time step are: Buffer Mass:

 $m_{B} \ddot{u}_{B} + P_{S} - P_{B}$ (4)
Beam: $m_{A} \ddot{u}_{A} = m_{A}g + F_{RJ} - P_{J} - 2P_{S}$ (5)
Upper Pipe: $m_{P} \ddot{u}_{P} = m_{P} g + F_{RJ} - P_{A}$ (6)
Loading Mass: $m_{C} \ddot{u}_{C} = m_{C} g - F_{RJ} + P_{J}$ (7)

The loading mass includes the mass of the collar zone pipe. In the test facility, there are load cells between the beam and the anchor zone pipe. The buffer mass is currently assumed to be the mass of the piston. It is possible to increase the buffer mass and to modify the buffer response at and subsequent to impact.

3.4 Method of solution

In order to solve the generally non-linear response of the system after impact, there are several techniques that have been used or are being evaluated. These range from a simple finite difference application of Newton's second law to Newmark's method (e.g. Chopra 1995). The finite difference approach assumes that forces (and consequently accelerations) do not vary during the time increment On the other hand, Newmark's method assumes that the changes in acceleration ($\Delta \ddot{u}_t$) and velocity ($\Delta \ddot{u}_t$) during a calculation time interval (A_t) may be expressed in terms of the change in displacement ($\Delta \ddot{u}_t$). The form of these relations are:

$$\Delta \ddot{\mathbf{u}}_{t} = \frac{\Delta u_{t}}{\beta (\Delta t)^{2}} - \frac{\dot{u}_{t}}{\beta \Delta t} - \frac{\ddot{u}_{t}}{2\beta}$$
⁽⁸⁾

$$\Delta \dot{\mathbf{u}}_{t} = \frac{\gamma \Delta \mathbf{u}_{t}}{\beta \Delta t} - \frac{\gamma \dot{\mathbf{u}}_{t}}{\beta} + \Delta t \left(1 - \frac{\gamma}{2\beta} \right) \ddot{\mathbf{u}}_{t}$$
(9)

where \ddot{u}_t and \ddot{u}_t are, respectively, the acceleration and the velocity at the start of the time increment.

The differential form of the equation of motion for a mass (m) interacting with a spring with stiffness (k) and dashpot with damping coefficient (c) is:

$$\mathbf{m}\,\Delta\ddot{\mathbf{u}}_{1}\,+\,\mathbf{c}\,\Delta\dot{\mathbf{u}}_{t}\,+\,\mathbf{k}\,\Delta\mathbf{u}_{t}\,=\,\Delta\mathbf{P}\tag{10}$$

where ΔP is the change of external force.

Substituting from Equation 8 and Equation 9 into Equation 10 and re-arranging gives:

$$\Delta u_{t} = \frac{m\left(\frac{\dot{u}_{t}}{\beta\Delta t} + \frac{\ddot{u}_{t}}{2\beta}\right) + c\left(\frac{\gamma\dot{u}_{t}}{\beta} + \Delta t\left(1 - \frac{\gamma}{2\beta}\right)\ddot{u}_{t}\right)}{\frac{m}{\beta(\Delta t)^{2}} + \frac{c\gamma}{\beta\Delta t} + k}$$
(11)

where $\gamma = 1/2$

and $\beta = 1/4$ constant acceleration

or $\beta = 1/6$ linear change of acceleration.

Further, equation 11 can be considered to be of the form $\Delta u_t = \Delta P/K$, where ΔP is equivalent to an out-of-balance force and K is equivalent to an instantaneous stiffness of response to movement at the start of the time increment.

The simultaneous equations of motion (Equations 4 to 7) can be represented in matrix form as:

$$\begin{bmatrix} 2\mathbf{K}_{\mathbf{B}} & -2\mathbf{k}_{\mathbf{S}} & \mathbf{0} & \mathbf{0} \\ -2\mathbf{k}_{\mathbf{S}} & \mathbf{K}_{\mathbf{A}} & -\mathbf{k}_{\mathbf{A}} & \mathbf{0} \\ \mathbf{0} & -\mathbf{k}_{\mathbf{A}} & \mathbf{K}_{\mathbf{P}} & -\mathbf{k}_{\mathbf{R}} \\ \mathbf{0} & \mathbf{0} & -\mathbf{k}_{\mathbf{R}} & \mathbf{K}_{\mathbf{R}} \end{bmatrix} \begin{bmatrix} \Delta \mathbf{u}_{\mathbf{B}} \\ \Delta \mathbf{u}_{\mathbf{A}} \\ \Delta \mathbf{u}_{\mathbf{P}} \\ \Delta \mathbf{u}_{\mathbf{P}} \end{bmatrix} = \begin{bmatrix} 2\Delta \mathbf{P}_{\mathbf{B}} \\ \Delta \mathbf{P}_{\mathbf{A}} \\ \Delta \mathbf{P}_{\mathbf{P}} \\ \Delta \mathbf{P}_{\mathbf{C}} \end{bmatrix}$$
(12)

where $\mathbf{K}'_{\mathbf{B}}, \mathbf{K}'_{\mathbf{A}}, \mathbf{K}'_{\mathbf{P}}$ and $\mathbf{K}'_{\mathbf{C}}$ are the equivalent instantaneous stiffnesses for the buffer, beam, upper pipe and loading mass, respectively, given by:

$$\mathbf{K}_{\mathbf{B}}^{'} = \frac{\mathbf{m}_{\mathbf{B}}}{\beta(\Delta t)^{2}} + \mathbf{k}_{\mathbf{B}} + \mathbf{k}_{\mathbf{S}} + \frac{\mathbf{c}_{\mathbf{S}}\gamma}{\beta\Delta t}$$
(13)

$$\dot{\mathbf{K}_{A}} = \frac{\mathbf{m}_{A}}{\beta(\Delta t)^{2}} + \mathbf{k}_{A} + \frac{\mathbf{c}_{A}\gamma}{\beta\Delta t} + 2\mathbf{k}_{S} + \frac{2\mathbf{c}_{S}\gamma}{\beta\Delta t}$$
(14)

$$\mathbf{K}_{\mathbf{P}}' = \frac{\mathbf{m}_{\mathbf{P}}}{\beta(\Delta t)^2} + \mathbf{k}_{\mathbf{R}} + \frac{\mathbf{c}_{\mathbf{R}}\gamma}{\beta\Delta t} + \mathbf{k}_{\mathbf{A}} + \frac{\mathbf{c}_{\mathbf{A}}\gamma}{\beta\Delta t}$$
(15)

$$\mathbf{K}_{\mathbf{C}} = \frac{\mathbf{m}_{\mathbf{C}}}{\beta(\Delta t)^2} + \mathbf{k}_{\mathbf{R}} + \frac{\mathbf{c}_{\mathbf{R}}\gamma}{\beta\Delta t}$$
(16)

and k_S , k_C and k_R are the instantaneous stiffnesses and c_S , c_C and c_R are the damping coefficients of the buffer/beam contact surface, anchor load cell and the reinforcement system responses, respectively, given by:

$$\dot{\mathbf{k}s} = \mathbf{k}s + \frac{\mathbf{c}s\gamma}{\beta\Delta t}$$
 (17)

$$\mathbf{k}_{\mathbf{A}} = \mathbf{k}_{\mathbf{A}} + \frac{\mathbf{c}_{\mathbf{A}}\gamma}{\beta\Delta t}$$
(18)

$$\dot{\mathbf{k}_{\mathrm{R}}} = \mathbf{k}_{\mathrm{R}} + \frac{\mathbf{c}_{\mathrm{R}}\gamma}{\beta\Delta t} \tag{19}$$

and $\ddot{A}P_B$, $\ddot{A}P_A$, $\ddot{A}P_P$ and $\ddot{A}P_C$ are the notional force changes for each component during the time increment given by:

$$\Delta P_{B} = m_{B} \left(\frac{\dot{u}_{Bt}}{\beta \Delta t} + \frac{\ddot{u}_{Bt}}{2\beta \Delta t} \right) - \frac{c_{S} \gamma}{\beta} (\dot{u}_{At} - \dot{u}_{Bt})$$
(20)
$$+ c_{R} \Delta t \left(1 - \frac{\gamma}{2\beta} \right) (\ddot{u}_{At} - \ddot{u}_{Bt})$$
(21)
$$\Delta P_{A} = m_{A} \left(\frac{\dot{u}_{At}}{\beta \Delta t} + \frac{\ddot{u}_{At}}{2\beta \Delta t} \right) - \frac{c_{A} \gamma}{\beta} (\dot{u}_{Pt} - \dot{u}_{At}) + \frac{2c_{S} \gamma}{\beta} (\dot{u}_{At} - \dot{u}_{Bt})$$
(21)
$$+ c_{A} \Delta t \left(1 - \frac{\gamma}{2\beta} \right) (\ddot{u}_{Pt} - \ddot{u}_{At}) - 2c_{S} \Delta t \left(1 - \frac{\gamma}{2\beta} \right) (\ddot{u}_{At} - \ddot{u}_{Bt})$$
(21)
$$\Delta P_{P} = m_{P} \left(\frac{\dot{u}_{Pt}}{\beta \Delta t} + \frac{\ddot{u}_{Pt}}{2\beta \Delta t} \right) - \frac{c_{R} \gamma}{\beta} (\dot{u}_{Ct} - \dot{u}_{Pt}) + \frac{c_{A} \gamma}{\beta} (\dot{u}_{Pt} - \dot{u}_{At})$$
(22)
$$+ c_{R} \Delta t \left(1 - \frac{\gamma}{2\beta} \right) (\ddot{u}_{Ct} - \ddot{u}_{Pt}) - 2c_{A} \Delta t \left(1 - \frac{\gamma}{2\beta} \right) (\ddot{u}_{Pt} - \ddot{u}_{At})$$
(23)
$$+ c_{R} \Delta t \left(1 - \frac{\gamma}{2\beta} \right) (\ddot{u}_{Ct} - \ddot{u}_{Pt})$$
(23)

The solution of Equation 12 by either Gaussian elimination or matrix inversion results in estimates of the incremental displacements that are added to the accumulated

displacements for each of the components. Further, the changes in the velocities are calculated for each component using Equation 8 and are added to the velocities at the start of the time interval. The accelerations for each component at the end of the time step are calculated using the forces estimated at the end of the time step.

3.5 Demonstration of simulation software for a yielding system

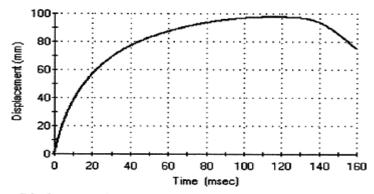
The solution technique has been implemented in software developed using Microsoft[®] Visual Basic. There is a main user interface which is divided into input specification and display of results. The input interface allows for specification of:

- The reinforcement system (i.e. selection of type of reinforcement system with a predefined force displacement response such as shown in Figure 4, with or without pretension). Following specification, a summary is displayed of the key properties such as force, displacement and energy absorption capacities.
- The loading mass (i.e. mass, height of drop, velocity of impact, nominal input energy).
- The buffer configuration. (i.e. piston pre-compressed or additional mass added).
- The anchor configuration (i.e. analysis of separate components or the upper pipe, beam and buffer combined).
- Execution control (i.e. analysis type, time increment, number of iterations, total duration).

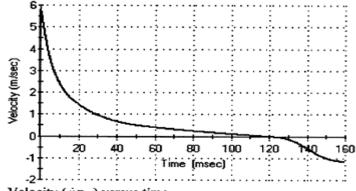
For the demonstration analysis, the following input variables were used:

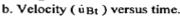
- Reinforcement system response given in Figure 4, pretensioned to 50 kN.
- Loading mass $M_C=2040$ kg.
- Drop height 1850 mm (impact velocity 6.02 m/s).
- Notional impact energy of loading mass 37 kJ.
- Beam mass M_A =645 kg and anchor pipe M_P = 30 kg.
- Combined beam and anchor pipe.
- Simple application of Newton's second law with time increment of 10 μ S and total execution time of 160 ms.

Following execution, detailed and summary information is available for all components of the simulated test (i.e. buffers, beam, loading mass and reinforcement system). For example, Figure 7 shows the buffer displacement, velocity and acceleration variations with time during the simulated test. Figure 8 shows the variations of displacement, velocity and acceleration for the reinforcement system together with the force variation with time. Figure 9 shows the reinforcement force-displacement response and Figure 10 shows the variations of component energies with time. For this particular illustrative example, it is easy to see that the reinforcement system is providing a uniform force resistance over a large range of



a. Displacement (uBt) versus time.





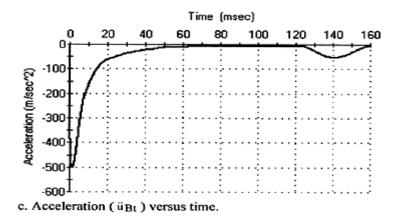


Figure 7. Summary data for beam/buffer response after execution of simulation software.

displacement, as required for a reinforcement system to sustain high energy impacts.

An important feature of the energy-time chart is that, for the simulation shown, the reinforcement energy absorption is ~35 kJ compared with the nominal mass energy of 37 kJ at impact. It is also worth noting that additional energy (denoted in Figure 10 as Input) is associated with vertical downward motion of the mass. This additional energy has to be absorbed by the reinforcement to bring the mass to rest.

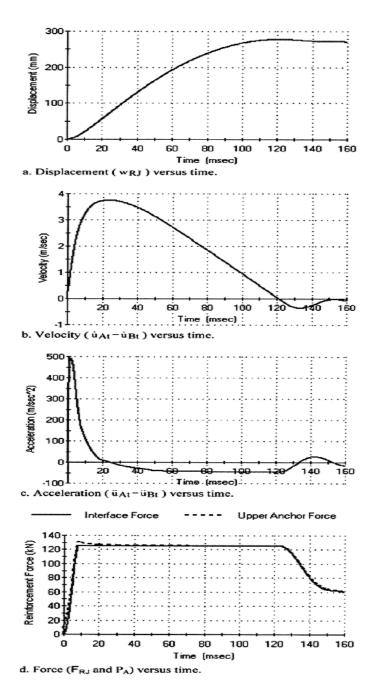
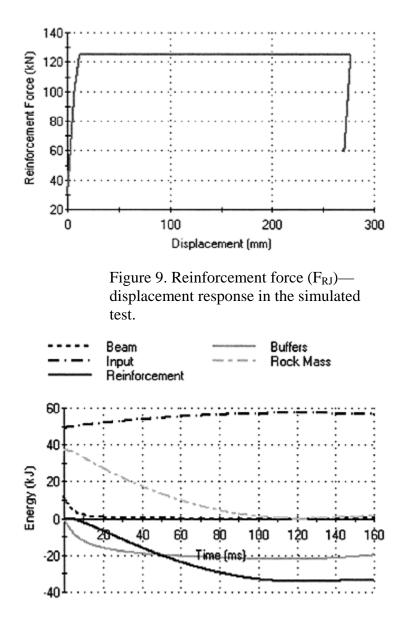
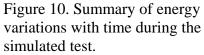


Figure 8. Summary data for reinforcement system response after execution of simulation software.





4 DATA ANALYSIS METHODOLOGY

The data analysis methodology consists of three main stages:

- 1. Reviewing and selecting data for analysis.
- 2. Filtering of the selected data.
- 3. Analysis of the filtered data over a selected time interval.

This methodology has been incorporated into the software developed using Microsoft[®] Visual Basic, the same programming language used to develop the simulation software described and demonstrated in Section 3. The software consists of interactive user interfaces with the data displayed in charts that may be "windowed" and zoomed to review data. The results used to demonstrate the analysis were obtained in a test similar to the one simulated earlier. In summary, loading mass (M_C) is 2040 kg, drop height 1850 mm, impact velocity 6.02 m/s and nominal impact energy associated with the mass of 37 kJ.

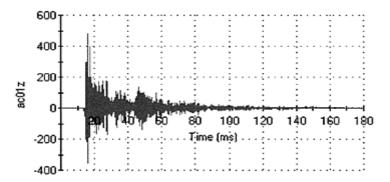


Figure 11. Acceleration-time plot from the accelerometer on the beam above the buffer.

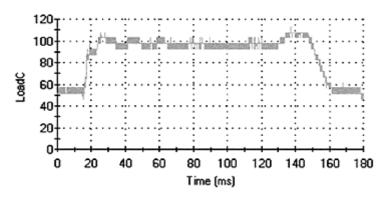


Figure 12. Force-time plot from the collar load cell.

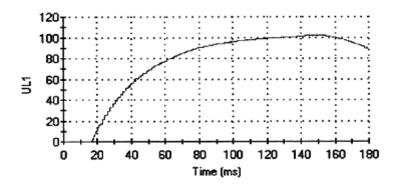


Figure 13. Displacement-time plot from the motion sensor.

4.1 Testing data

Data were collected from \sim -120 ms to 700 ms, where the datum (time=0) is the time when the breaking of a laser beam causes data storage. The data excess to the requirements for analysis are ignored by selecting a time range window on a chart of the data.

Data obtained during the test are shown in:

- Figure 11 (beam acceleration-time).
- Figure 12 (collar force-time).
- Figure 13 (beam/buffer displacement-time).

In these and subsequent figures, the range of useful data ranges from approximately initial impact

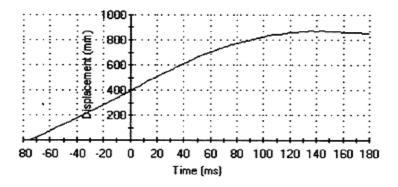


Figure 14. Displacement-time plot for the loading mass derived from the video recording.

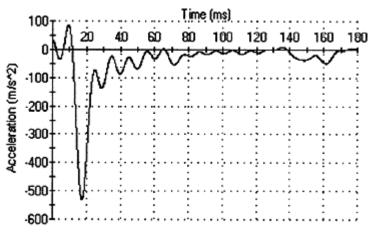


Figure 15. Beam acceleration-time plot corresponding to Figure 11 after filtering out frequencies above 100 Hz.

(at time ~15 milliseconds) and after the test has finished (time ~180 milliseconds).

4.2 Analysis of video recording data

The software included with the video recording system allows selected points on the test specimen to be tracked with time. The raw data are corrected for distortion due to the vertical plane of the test being at an angle to the camera recording plane. Typically, a point on either the end of the reinforcement or a point on the loading mass (e.g. Figure 14) are tracked. These data are synchronised with the other measurements and added to the data available for analysis.

4.3 Fast Fourier Transforms for frequency analysis and flltering of raw data

It was recognised from previous testing that filtering of these data would be required to interpret the results in a meaningful way. Initially, it was anticipated that frequency analysis using a Fast Fourier Transform (FFT) and Inverse Fast Fourier Transform (IFFT) for frequencies below a certain threshold would be used. This has been found to be valid for the accelerometers and load cells. For example, Figure 15 shows the data

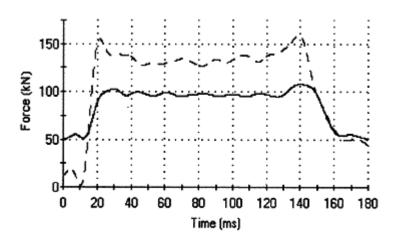


Figure 16. Collar force (P_{EP}) -time plot corresponding to Figure 12 after filtering out frequencies above 150 Hz.—anchor force (P_A) also shown as dashed line for comparison.

from the beam accelerometer filtered to eliminate all frequencies above 100 Hz and inverted to reflect the correct sense of acceleration. Figure 16 shows the data from the collar and upper anchor pipe load cells filtered to eliminate all frequencies above 150 Hz.

Integrating the filtered acceleration-time data to obtain velocity-time and displacement-time data is found to give acceptable results. However, it was quickly recognised that, even after filtering, attempting to differentiate displacement-time data to obtain velocity-time data and to then differentiate the resulting velocity-time data to obtain acceleration-time data would not be possible.

After an extensive review of techniques available for filtering time varying data, the Kalman filter was chosen as being most likely to satisfy the requirements for analysing the type of displacement data being obtained from the motion sensor and derived from the video recordings.

4.4 Kalman filter for multiple variables

The Kalman filter is capable of taking displacement-time data and predicting the filtered displacement-time data as well as the velocity-time data and the acceleration-time data. An added benefit of the Kalman filter is its ability to be able to incorporate additional information (e.g. acceleration-time data) from another sensor to improve the overall filtered response.

A further incentive to adopt the Kalman filter was that it was available as a component of a software library (Newcastle Scientific 2002) compatible with Microsoft[®] Visual Basic.

The Kalman filter is similar to a least squares fitting to data except that measurements can be incorporated into the fitting coefficients one at a time rather than all at once as with the standard least squares technique.

The Kalman filter propagates and updates what are known as the state vector and covariance matrix. For the dynamic test facility, the state vector comprises the displacement, velocity and acceleration associated with a component of the system. The covariance matrix consists of terms associated with the errors in the measurements of the state vector variables.

Input to the filter comprises:

- 1. The vector of measurements (one, two or three of displacement, velocity and acceleration).
- 2. The measurement noise matrix.
- 3. The matrix of partial derivatives of the measurements with respect to the states.
- 4. The propagation matrix for displacement, velocity and acceleration.
- 5. The current covariance matrix.

6. The process noise matrix.

The terminology and the formation of the various vectors and matrices are given in the following sections.

The vectors and matrices have the following dimensions:

N_S=number of state variables (3)

N_M=number of measurements (1 or 2)

4.4.1 State variables

The vector of state variables [X] has length N_s and is given by:

$$\begin{bmatrix} \mathbf{X} \end{bmatrix}^{\mathsf{T}} = \begin{bmatrix} \mathbf{x} & \dot{\mathbf{x}} & \ddot{\mathbf{x}} \end{bmatrix}$$
(24)

where x=displacement

 $\dot{\mathbf{X}}$ =velocity $\ddot{\mathbf{X}}$ =acceleration

4.4.2 Vector of measurements

The vector of measurements [Z] has length N_M . If measurements are made corresponding to all three state variables, then:

$$[\mathbf{Z}]^{\mathsf{T}} = \begin{bmatrix} \mathbf{z} & \dot{\mathbf{z}} & \ddot{\mathbf{z}} \end{bmatrix}$$
(25)

However, typically measurements are available for displacement alone, acceleration alone or displacement and acceleration. That is, no direct measurements of velocity are made.

4.4.3 Matrix of partial derivatives

The matrix [H] of partial derivatives of measurement variables with respect to the state variables has dimensions N_M by N_S . If the number of measurements corresponds to the number of state variables, [H] is given by:

$$[\mathbf{H}] = \begin{bmatrix} \frac{\partial \mathbf{z}}{\partial \mathbf{x}} & \frac{\partial \mathbf{z}}{\partial \dot{\mathbf{x}}} & \frac{\partial \mathbf{z}}{\partial \ddot{\mathbf{x}}} \\ \frac{\partial \dot{\mathbf{z}}}{\partial \mathbf{x}} & \frac{\partial \dot{\mathbf{z}}}{\partial \dot{\mathbf{x}}} & \frac{\partial \dot{\mathbf{z}}}{\partial \ddot{\mathbf{x}}} \\ \frac{\partial \ddot{\mathbf{z}}}{\partial \mathbf{x}} & \frac{\partial \ddot{\mathbf{z}}}{\partial \dot{\mathbf{x}}} & \frac{\partial \ddot{\mathbf{z}}}{\partial \ddot{\mathbf{x}}} \end{bmatrix}$$
(26)

For the analysis of data in this dynamic test facility, the following typical cases occur: – Measurement of displacement only:

$$[H] = [1 \ 0 \ 0] \tag{27}$$

- Measurement of acceleration only:

$$[H] = [0 \ 0 \ 1] \tag{28}$$

- Measurement of displacement and acceleration:

$$[H] = \begin{bmatrix} 1 & 0 & 0 \\ 0 & 0 & 1 \end{bmatrix}$$
 (29)

4.4.4 Propagation matrix

The N_S by N_S propagation matrix (Φ) is given by:

$$\left[\Phi \right] = \begin{bmatrix} 1 & \Delta t & 0.5 \Delta t^2 \\ 0 & 1 & \Delta t \\ 0 & 0 & 1 \end{bmatrix}$$
 (30)

4.4.5 Process noise matrix

The process noise matrix [Q] is N_S by N_S . Some experimentation is being used to estimate appropriate values for the components of this matrix.

4.4.6 Covariance matrix

The covariance matrix [P] is N_S by N_S . Again, some experimentation is being used to estimate appropriate values for its components.

4.4.7 Measurement noise matrix

The N_M by N_M measurement noise matrix [R] can be estimated by knowing the response characteristics for the particular sensor and how this might affect the precision of the measurement.

4.4.8 Summary

Mathematically, the operation of the Kalman filter may be described by the following 5 steps:

1 State Extrapolation—propagates the state vector to the time of the current measurement.

$$[X-]_{t} = [\Phi]_{t-1} [X+]_{t-1}$$
(31)

2 Error Covariance Extrapolation—propagates the covariance matrix to the time of the current measurement and adds process noise.

$$\begin{bmatrix} \mathbf{P} - \end{bmatrix}_{t} = \begin{bmatrix} \mathbf{\Phi} \end{bmatrix}_{t-1} \begin{bmatrix} \mathbf{P} + \end{bmatrix}_{t-1} \begin{bmatrix} \mathbf{\Phi} \end{bmatrix}_{t-1}^{\mathsf{T}} + \begin{bmatrix} \mathbf{Q} \end{bmatrix}_{t-1}$$
(32)

3 Kalman Filter Gain—is the gain weighting matrix given by:

$$\begin{bmatrix} \mathbf{K} \end{bmatrix}_{t} = \begin{bmatrix} \mathbf{P} - \end{bmatrix}_{t} \begin{bmatrix} \mathbf{H} \end{bmatrix}_{t}^{T} \begin{bmatrix} \begin{bmatrix} \mathbf{H} \end{bmatrix}_{t} \begin{bmatrix} \mathbf{P} - \end{bmatrix}_{t} \begin{bmatrix} \mathbf{H} \end{bmatrix}_{t}^{T} + \begin{bmatrix} \mathbf{R} \end{bmatrix}_{t}^{-1}$$
(33)

4 Error Covariance Update

$$[\mathbf{P}+]_{t} = [\mathbf{I}] - [\mathbf{K}]_{t} [\mathbf{H}]_{t} [\mathbf{P}-]_{t}$$
(34)

5 State Vector Update—updates the state vector with the current measurement, weighted by the gain matrix.

$$[\mathbf{X}+]_{t} = [\mathbf{X}-]_{t} + [\mathbf{K}]_{t} [[\mathbf{Z}]_{t} - [\mathbf{H}]_{t} [\mathbf{X}-]_{t}]$$
(35)

In these equations, the "–" and "+" signs in the vector [X] and matrix [P] indicate the initial and final estimated values, respectively.

4.5 Demonstration of Kalman filter

The data shown in Figure 13 was input to the Kalman filter routine together with the known (assumed) initial conditions for the beam (i.e. velocity=6.02 m/s and acceleration=9.81 m/s²). The filtered results for the displacement, velocity and acceleration variations with time are shown in Figure 17.

4.6 Engineering calculations

4.6.1 Forces and displacements

For the purposes of the calculations, the anchor zone components are lumped together. Following filtering of the data, it can be assumed that at any time the accelerations of the beam (\ddot{u}_A) and the loading mass (\ddot{u}_C) are known.

The net force on the loading mass (F_C) at any time is given by: $F_C=m_C\ddot{u}_C=m_Cg=F_{RJ}$

(37)

(38)

Then

$$F_{RJ}=m_C(g=\ddot{u}_C)$$

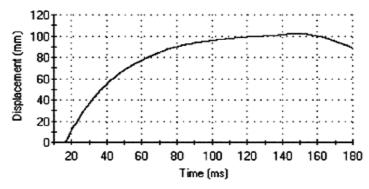
The net force on the beam (F_A) at any time is given by:

 $F_{A}=m_{A}\ddot{u}_{A}=m_{A}g+F_{RJ}-2(F_{B}-m_{B}g)$

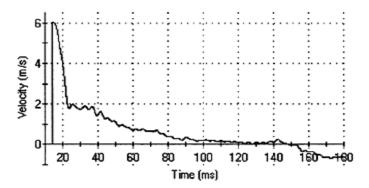
from which the buffer force may be estimated from

 $P_{\rm B} = (m_{\rm A}g + m_{\rm C}(g - \ddot{u}_{\rm C}) + 2m_{\rm B}g - m_{\rm A}\ddot{u}_{\rm A})/2$ (39)

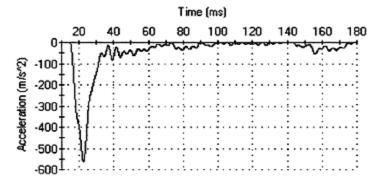
Also, by integrating the measured accelerations, the velocities ($^{\hat{u}_{C}}$ and $^{\hat{u}_{A}}$) are obtained. The displacement



a. Filtered displacement (uA) versus time.



b. Predicted velocity (uA) versus time.



c. Predicted acceleration (üA) versus time.

Figure 17. Filtered displacement, velocity and acceleration derived by Kalman filter from data shown in Figure 13.

of the beam (u_A) has been measured directly by the motion sensor and the mass displacement (u_C) estimated from the video recording. The reinforcement displacement (u_R) at the interface between the collar and anchor pipes is then given by:

 $u_{R}=u_{C}-u_{A} \tag{40}$

At this stage in the analysis, the variations of all varithat in some instances there may be a direct measureables during the test have been estimated. Also note ment corresponding to the estimate. For example, the load cells give measurements approximating to F_{RJ} . In general, the load cells will record different forces. The force measured by the anchor load cells includes the inertia force of the upper pipe (see Equation 6). The load cell at the collar measures the force between the reinforcement system plate and the loading mass. This collar force may be different from the reinforcement force at the interface due to load transfer from the reinforcement system to the pipe in the collar zone.

4.6.2 Momentum

The momentum of the components can be calculated at any time during the test. The changes in momentum can be assessed relative to the changes in external forces acting on the components.

4.6.3 Energy

The kinetic energy of the components of the system (beam and loading mass) and the energy absorbed by the reinforcement and buffers may be calculated at any time during the test and an "energy balance" calculation performed.

The energy of the beam (E_A) at any time is given by:

$$E_A = m_A v_A^2/2$$
(41)

Similarly, the energy of the loading mass (E_C) at any time is given by: $E_C = m_C v_C^2/2$

(42)

(44)

The energy absorbed by the reinforcement (E_R) is given by: $E_R = \int F_{RJ} dw_{RJ}$

(43)

And the energy absorbed by the buffers (E_B) is given by: $E_B=\int P_B du_B$

In addition, using the buffer piston as a reference height, after impact it is assumed that additional kinetic energy is gained by loss of potential energy which in total is given by:

$$E_{P}=m_{A} u_{A}+m_{B} u_{B}+m_{P} u_{B}$$
(45)
The net energy E_{N} at any time is given by:

$$E_{N}=E_{A}+E_{C}+E_{P}-E_{R}-E_{B}$$
(46)

(47)

where v_0 is the velocity of impact.

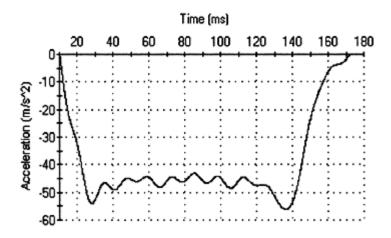


Figure 18. Acceleration (\ddot{u}_B) -time plot for the loading mass.

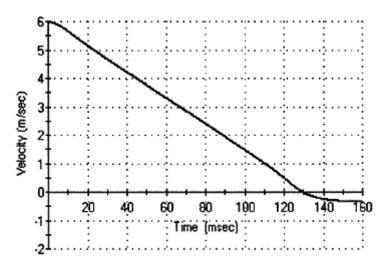


Figure 19. Velocity-time response of loading mass.

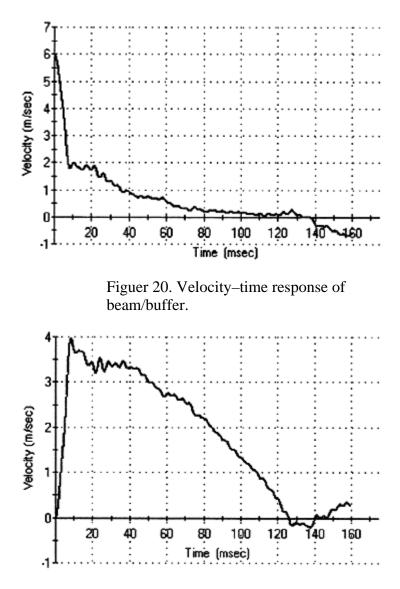
If the reinforcement system does not fail and the loading mass is brought to rest, then E_N should equal zero at the end of the test.

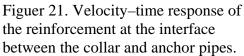
4.6.4 Detailed analysis of testing data

Following filtering of the results, a start and finish time are selected for detailed analysis. For this example, the range of data that is used for the engineering calculations is from 15 ms to 175 ms, to give the same time range (160 ms) of calculations that were used previously in the simulation of this test. The input data used for the detailed analysis of the test are:

- Beam/buffer displacement (Figure 17a).
- Beam/buffer velocity (Figure 17b).
- Beam/buffer acceleration (Figure 17c).
- Loading mass displacement (Figure 14).
- Loading mass acceleration (Figure 18).

The velocity of the loading mass obtained by integrating the acceleration is given in Figure 19. The difference between this velocity and the velocity of the beam/mass was used to derive the reinforcement velocity shown in Figure 21. The maximum velocity of reinforcement loading is \sim 4m/s and is slightly greater than that predicted by the simulation shown in Figure 8b.





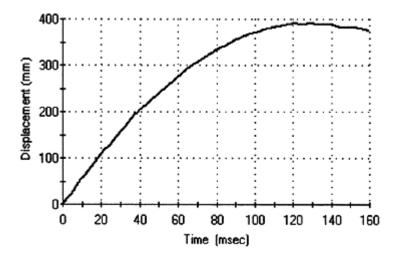


Figure 22. Displacement-time response of the loading mass.

Similarly, the displacements for the respective components are given in Figure 22, Figure 23 and Figure 24, respectively. The displacement between the collar and anchor pipes at the completion of the test was measured to be 283 mm. This compares well with the displacement of the reinforcement derived from both the analysis of the monitoring and recording system and the simulation given previously.

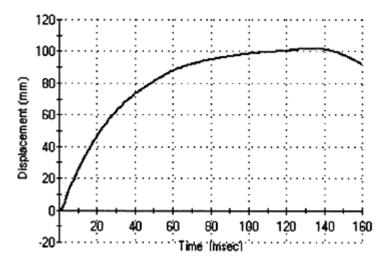


Figure 23. Displacement-time response of beam/buffer.

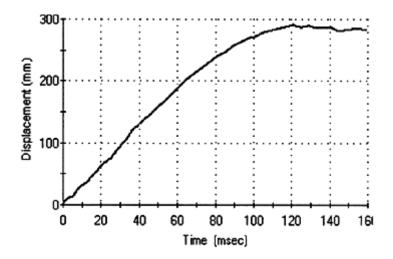


Figure 24. Displacement-time response of the reinforcement at the interface between the collar and anchor pipes.

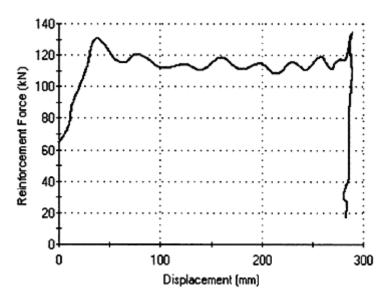
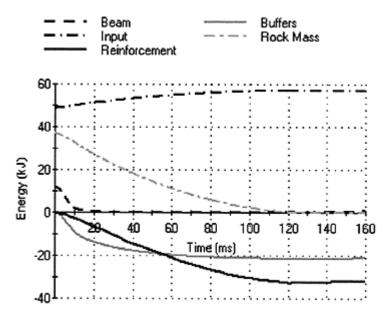
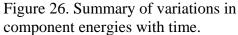


Figure 25. Force-displacement response of the reinforcement system at the interface between the collar and anchor pipes.

The force-displacement response for the reinforcement system is given in Figure 25. It is worth repeating that it is expected that the interface force (F_{RJ}) will lie between the measured upper anchor force (P_A) and collar force (P_{EP}) shown in Figure 16.





Finally, the energy variations of the components and the overall testing system are shown in Figure 26. These latter two results from the analysis of the test data compare favourably with the results of the simulation presented in Figure 9 and Figure 10, respectively. The energy absorbed by the reinforcement system is over \sim 32 kJ, which represents \sim 90% of the notional loading mass energy at impact (37 kJ). The energy absorbed by the buffers is approximately twice that of the initial kinetic energy associated with the beam.

5 FUTURE DEVELOPMENTS

The analysis of data described in the previous section used single instrument data as the basis for the estimates of the various variables associated with each component. In future, the analysis method will aim to better define the various estimates by "averaging" values derived from different instruments.

Planned future developments of the test facility will enable the simulation and testing of the combined effects of surface restraint and reinforcement in resisting dynamic loading.

6 CONCLUDING REMARKS

A new dynamic test facility has been established and commissioned. Simulation and data analysis techniques have formed integral components of the development of the facility and have aided understanding of how reinforcement responds to dynamic loading and the types and locations of instrumentation required to define forces and displacements within the system. With this information, the energy absorbed by the reinforcement can be quantified. The data analysis software enables rapid processing of data.

The preliminary testing program and associated simulations have demonstrated that significant impact velocities are generated. The reinforcement system must be capable of absorbing significant energies in order to bring the loading mass to equilibrium.

ACKNOWLEDGEMENTS

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7

Rockfalls and failure mechanisms

Controlling rockfall risks in Australian underground metal mines

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ABSTRACT: A comprehensive database of rockfall events is used to analyse the effectiveness of rock reinforcement and surface support in underground Australian metalliferous mines. It is found that one of the main shortcomings in the use of rock reinforcement and support in preventing rockfall injuries relates to the timing of installation of reinforcement and the deficient use of surface support. The risk profile of rockfalls with regards to causing injuries is found to be higher near active mining faces, where mining activities are intense and where mine workers are exposed to unsupported faces and walls. Away from active faces, rockfalls also commonly occur, but only a very small proportion of them causes injuries.

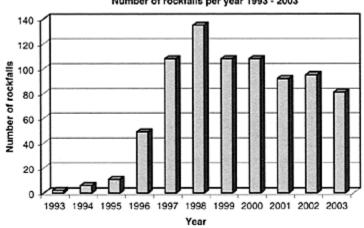
1 INTRODUCTION

Rockfalls remains one of the major sources of injuries and fatalities in underground metal mines (Potvin et al. 2003). A number of control measures can be applied to mitigate the risk associated with rockfalls. For example, exclusion zones, remote equipment and specific working procedures can assist in removing personnel from being exposed to rockfall hazards. Another widely used approach involves ground support and reinforcement, which mitigates rockfall risks by reducing the likelihood of rockfalls. In most mines, this is the control measure of choice. In fact, many countries have adopted regulations encouraging the use of some form of ground reinforcement in all mine excavations accessed by mine workers.

The cost of controlling the risk of rockfalls can be considerable. In difficult ground conditions, the direct cost of ground support and reinforcement can reach well over 25% of the total mining cost. This is notwithstanding other "hidden costs" related to increases in mining cycle time associated with extensive ground support activities, as well as any rehabilitation work that may be required in older excavations.

This paper will explore how effective ground reinforcement and support has been in controlling the consequences of rockfalls within the Australian mining industry over the last 10 years. The discussion relies on a comprehensive rockfall database assembled as part of the scope of a multi-phase research project currently being undertaken at the Australian Centre for Geomechanics, University of Western Australia.

The database contains 795 rockfall events from 29 mines. The rockfalls events have occurred between 1993 and 2003. The distribution of the number of rockfalls with time is shown in Figure 1. The reduction in the occurrence of rockfalls in recent years has also translated in a reduction in rockfall injuries, as can be seen in Figure 2. The increasing trend in the number of rockfalls between 1993 and 1997 is due to greater awareness and improvement in keeping records of rockfalls rather than a "real" increase in rockfall occurrence.



Number of rockfalls per year 1993 - 2003

Figure 1. Rockfall database summary of falls of ground in Australia: 1993-2003.

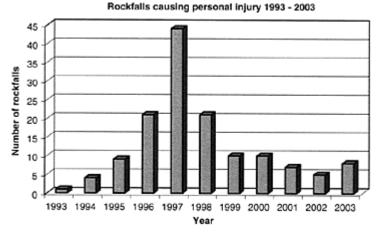


Figure 2. Chart showing rockfalls causing personal injury indicating a flattening trend over the past four years.

The rapid decrease of rockfalls injuries after 1998 shown in Figure 2 is interpreted as the result of changes in mining practices near the face during this period. It is inferred that a number of guidelines and regulations issued in the mid to late 1990's by the Department of Minerals and Energy of Western Australia (Lang & Stubley 2003) encouraging good ground control practices had a significant impact in changing practices, not only in Western Australia, but also in other States by informal influences (adopted by mines as "good practice" prior to formal implementation requirements by state regulators). Many mining companies have adopted safer practices such as installing ground support "incycle" rather than as on a several cuts "campaign" basis, increasing usage of surface support and mechanised techniques for installing ground reinforcement, thereby reducing personal exposure to unsupported ground.

2 ROCKFALL RISK PROFILE

Rockfall risks can be characterised as a combination of the likelihood of occurrence and the exposure of the personnel to potential falls of ground. To develop an understanding of the risk profile, it is useful to consider where in the mine the rockfalls occurred, as the likelihood and the exposure are location dependant. It is not surprising that the risk of rockfall is highest near the active face (refers to new development of excavations that mine employees can access, work or travel in), where the mining activity is intense and the ground has been disturbed by recent blasting. Furthermore, some mining tasks require personnel to be exposed to the advancing mining face, which is rarely supported.

The higher risk of rockfall near the face is demonstrated in Figure 3, with 81% of rockfall injuries occurring within 10 m of an active mining face.

To understand the causes of rockfalls near the face, one not only needs to assess the effectiveness of the

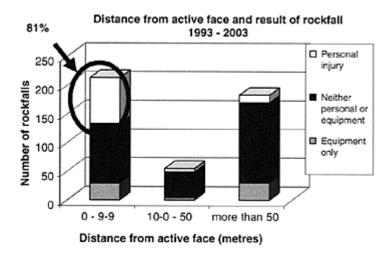


Figure 3. Result of rockfall in relation to the distance the incident occurred from an active mining face.

Rockfalls in unsupported ground causing personal injuries 1993 - 2003

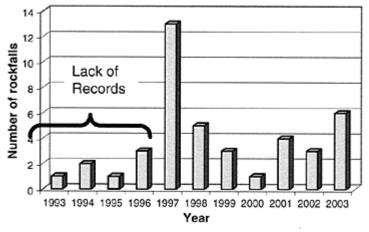


Figure 4. Summary of falls of ground that resulted in personal injury.

ground control measures but also the details of the ground support installation process. In fact, the data shows that in the earlier years (1993–1997) covered by the study, a large proportion of rockfall injuries occurred before the ground reinforcement was installed.

Figure 4 shows an annual distribution of rockfalls causing injuries, that have occurred before the reinforcement and support was installed. These will be referred to as "unsupported rockfall" injuries. It could be inferred from this graph, that because the "unsupported rockfall" injuries have been reduced dramatically since 1998, the changes in mining practices preventing the exposure of personnel to unsupported ground have resulted in a significant improvement in minimising rockfall injuries.

The data has also shown clearly that the smaller rockfalls are the ones causing most injuries. Figure 5 is a distribution of rockfalls according to their weights. Over 90% of rockfalls resulting in personnel injuries are located within the first column of the graph, which represents rockfalls that are less than 2 tonne.

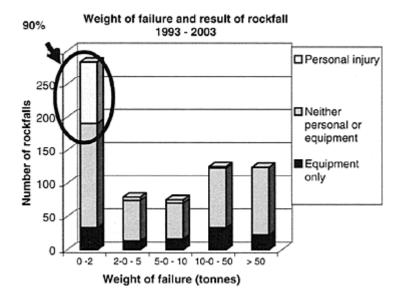


Figure 5. Chart highlighting most personal injuries are caused by weights of failure less than 2 tonne.

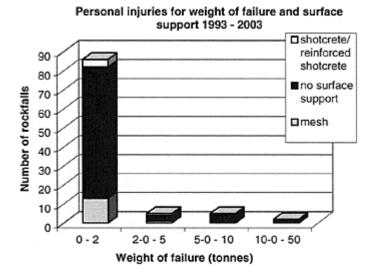


Figure 6. Chart showing low weight of failure rockfalls that may have been prevented by the installation of surface support.

It is commonly understood that whilst the role of rock reinforcement is primarily to prevent movement and deformation of the rock mass (aiming to prevent larger rockfalls), the smaller rocks that may detach from the excavation boundaries should be retained using some form of surface support. The large incidence of small rockfalls causing injuries could therefore be indicative of a deficient use of surface support. This assumption is supported by the database as shown in Figure 6, with a large proportion of the data in the first column representing rockfalls injuries that had no surface support (shown in black).

One of the major conclusions emerging from this research is that over the last 10 years, the main failure of Australian ground stabilisation techniques in terms of preventing rockfall injuries in mines has more to do with the timing of installation of the reinforcement and the deficient usage of surface support, than any other issues.

3 CURRENT GROUND SUPPORT PRACTICES IN AUSTRALIAN MINES

The following section will consider some of the ground support practices currently used in Australia and how changing techniques over the past decade have helped to reduce the number of rockfall related incidents near active mining faces. Jumbos perform a number of tasks within the development mining cycle at most operations around Australia. This may include mechanical scaling, boring blastholes, boring ground reinforcement holes, installing ground reinforcement and mesh. This multi-tasking minimises the travel time associated with equipment movement, reduces capital costs, and eliminates the time wasted in rigging up and down associated with utilising a number of different units to complete the work.

Mechanical scaling using jumbos is an accepted and popular practice in Australia. It is perceived by many as being safer than manual scaling, despite the fact that jumbos are not specifically designed to carry out scaling activities. There is on-going discussion within the industry as to whether manual scaling is required to "complete" scaling tasks, or conversely, whether it should only be carried out by the most experienced and highly trained mining personnel. This is usually decided on a mine-by-mine basis by mining technical and operational staff that have carried out an assessment of the suitability of the mechanical scaling process with respect to the relevant site-specific conditions.

Jumbos will drill blast hole patterns as part of their normal duties. They will also often drill ground reinforcement hole patterns. As a consequence, consideration has to be given to some important practical issues. The use of "regular" length slides (for drilling faces) may require headings heights to be sufficient to allow a vertical or near vertical holes to be drilled for reinforcement installation. Otherwise, reinforcement boreholes will be drilled inclined so the slide fits within the heading. This is not considered good ground reinforcement practice, as the depth of coverage of reinforcement is reduced and the bolt is more likely to be submitted to an increased degree of shearing. A reluctance to provide extra heading height is usually a result of the impact on the development cycle and the additional cost and time taken in the removal of extra development ore or waste rock.

An alternative is to fit the jumbo with either shorter slides or use "split-feed" slides. The common criticisms regarding reducing the slide length or the use of split feed slides may include the reduction in excavation advance per drilled round or an increase in maintenance activity duration respectively. In either case, a decrease in productivity or increase in maintenance costs may result.

Friction Rock Stabilisers FRS (or split sets) are extensively used in both the walls and backs of many Australian mines. Undeniably, the FRS owe their popularity to their ease of installation by jumbo drills, which in turn, minimises the exposure of personnel to rockfall hazards. Whilst the blanket application of this type of support may not be universally recognised as good practice, FRS were often used in the past because they are a "one pass" support system (i.e. offering immediate support without the need for post-grouting or tightening of bearing plates). Until recent years, the FRS was the only "one-pass" rock bolt providing immediate support that could be efficiently and "remotely" installed with a jumbo.

Mechanisation of the meshing process using jumbos is a valuable tool in improving efficiency of development activities. Skilled operators can remotely "clasp" a sheet of mesh with a FRS bolt, "pin" it to the backs, thereby completing the installation of both the reinforcement and the surface support in a single pass. This proves to be a very efficient and safe method of operation that produces quality mesh installation. Figure 7 shows a series of photos illustrating the installation of mesh using a Jumbo in an Australian mine.

Despite all the advantages of FRS reinforcement with regards to productivity (onepass installation with a jumbo and assisting mechanisation of the installation process), the long-term suitability of this reinforcement remains an issue, as it may be more prone to corrosion than fully encapsulated reinforcement types. Furthermore, it generally has a lower load bearing capacity when compared to chemically bonded reinforcement. The problem is overcome at some mines by installing a second pass of grouted (often resin) bolts in designated permanent or long-term openings. Although this practice implies that the reinforcement



Photo 1





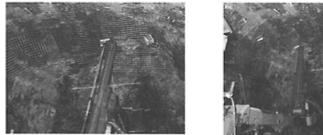








Figure 7. Sequence of photos illustrating the remote installation of mesh using a Jumbo. The photos show a sheet being pinned (1), turned across the excavation (2), completing the installation by installing reinforcement in the backs (3 and 4) respectively.

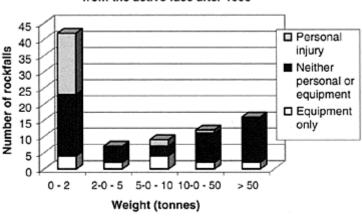
is "doubled up", the efficiency of jumbo installation of the primary support allows for the development cycle to be completed more rapidly without exposing mine workers to unsupported ground.

A number of mining companies recognised an opportunity to optimise the reinforcement process and reduce costs by removing the "first pass" installation of reinforcement in permanent openings by eliminating the FRS. To retain the productivity however, a chemical bolt that could be installed with a jumbo was required. In conjunction with reinforcement manufacturers, substantial advances in recent years have focused on developing a "one pass" resin based reinforcement products which can be easily installed with a jumbo. However, problems may still exist when attempting to install resin bolts and mesh in a single pass. This could be overcome by developing a system that addresses the practical difficulties which may involve considering the option of pinning the mesh with one or bolts in strategic locations and aligning the mesh sheet prior to installing the pattern of resin bolts.

4 REMNANT RISK OF ROCKFALLS

It has been previously established that significant changes in Australian mining practices have reduced the risk of rockfalls in areas within 10 metres of active faces. Looking at opportunities for further improvements, it is important to understand the remnant risks associated with the improved mining practices. Some insight can be gained by isolating the more recent data. The rest of this paper will therefore concentrate on rockfall events that have occurred after 1998.

Figure 8 indicates that despite improved practices, there are still a significant number of rockfalls covering the full range of sizes, occurring near the active face. There are also on average, approximately 5 to 6 injuries per annum at the face, and this rate has remained



Weight of failure and result of rockfall < 10m from the active face after 1998

Figure 8. Chart showing most personal injuries near the active face are caused by falls of ground less than 2 tonne (preventable by surface support).

consistent since 1999. It can be noted that most injuries (grey colour) are still caused by rockfalls with a weight of less than 2 tonne.

Figure 9 is a distribution of rockfall injuries according to their origin within the excavation. The graph clearly identifies that most injuries come from rockfalls detaching from the face and walls. Mining faces (and the lower section of the walls) generally have no surface support or reinforcement, unless poor ground, high stress or rockbursting conditions are identified as a hazard during development and the required reinforcement or surface support controls are installed.

To reduce this risk, one could consider reducing the exposure of people to the face by modifying some of the mining cycle tasks or alternatively, attempt to reduce the occurrence of those small rockfalls by improved scaling techniques, or by using systematic surface support in mining faces and walls. Some mines with severe ground control problems have already adopted this latter solution (Brenchley & Spies 2004).

In relation to the rockfall injuries originating from the backs, all but one case had reinforcement installed. The unsupported rockfall injury in fact occurred whilst installing ground support. This could be considered an isolated incident, and the cause may be associated with the installation procedure, inadequate scaling or a combination of the two. Four of the six other cases had no surface support and in all four cases, the workers were struck whilst working in close proximity to the face. Although these rockfalls originated from backs, they presumably occurred within half a metre of the face, in an area that for practical reasons, is rarely surface supported. These four cases could in fact be associated

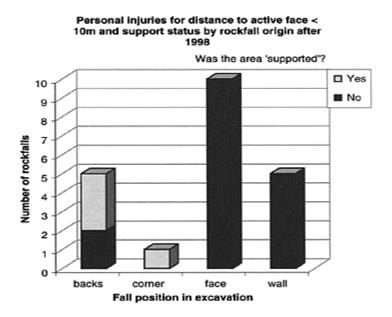


Figure 9. Most personal injuries associated with development activities occur near the face and can be attributed to a lack of support. with the 10 cases involving rockfalls injuries at the face, as they have similar causes and possibly similar remedial solutions.

Mining activities near the active face that are most at risk of rockfall injuries are shown in Table 1. Amongst the five most at risk activities, marking-up the face, cleaning blast holes and charging blast holes can be attributed to personnel exposure to unsupported advancing faces. This is consistent with statistics shown in Figure 9.

Table 1. Chart showing the activities being carried out near active mining faces that caused personal injuries.

Activity	No. of falls
Ground support—installing	4
Charging	3
Cleaning blast holes	3
Drilling—jumbo	3
Face mark up	3
Jumbo off-siding	1
Mapping	1
Scaling—hand	1
Scaling—mechanical	1
Unknown	1
Walking—development heading	1
Washing down	1

5 ROCKFALLS AWAY FROM ACTIVE FACES

Let us arbitrarily define a rockfall "away" from active faces being over 50 metres from an advancing development mining heading. The data demonstrates that the risk profile from rockfalls happening away from the face is very different than the ones occurring near the face. There are fundamental differences between the two datasets. Near the face, the area is relatively small (in the order of fifty square meters) and the exposure of people is intense due to near-continuous mining activities that are being carried out. The area away from the face (where workers have access), may comprise tens and sometimes hundreds kilometres of drives. Most of this area has no specific activities other than travelling personnel or equipment. Therefore, most locations away from the active mining face have a very low exposure for personnel.

Despite the fact that numerous rockfalls may occur away from the face, the chances of having personnel injured by them is relatively small and therefore, the risk can be characterised as being lower when compared to rockfalls near mining faces. This is supported by the database, as in terms of consequences, only 5 of the 168 rockfalls away from the face resulted in personal injuries. Figure 10, which shows activities near or exposed to rockfalls away from the face, also supports

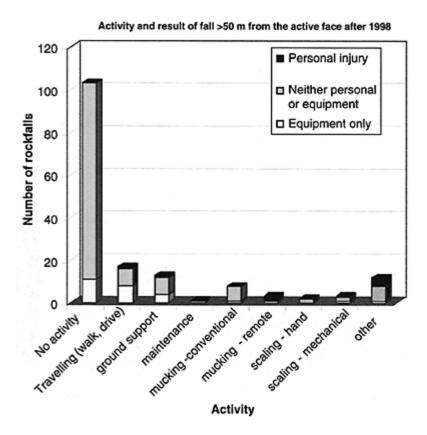


Figure 10. Chart showing most activities away from the face are not associated with personal injury. Only 5 cases out of a total of 168 cases resulted in a personal injury occurring.

the interpretation that these areas have a lower risk for rockfall injuries. The graph indicates that most of these rockfalls have occurred where there are no activities taking place. No specific activity away from the face seems more prone to rockfall injuries.

Lessons can be learned from investigating the causes of rockfalls, even when their consequences are marginal (i.e. no injuries). As with rockfalls near active faces, the lack of surface support appears to once again account for a significant proportion of the rockfalls away from the face. Figure 11 is a distribution of rockfall sizes and surface support practices. Assuming that mesh can retain up to 2 tonnes of broken rock, it can be

asserted that approximately 70% of the small rockfalls (less than 2 tonnes) could have been prevented by mesh. However, it may not always be practical to

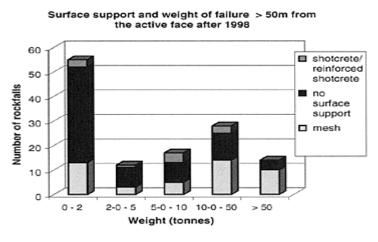


Figure 11. Chart highlighting that 70% of falls weighing 2 tonnes or less could have been prevented by the installation of surface support

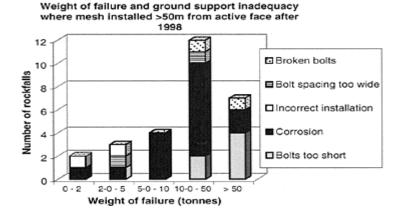


Figure 12. Failure inadequacies shown for the various weights of failure where the fall area has been meshed. Corrosion is the dominant support inadequacy for higher tonnage rockfalls. surface support an entire mine, especially when the mine infrastructure is extensive and old. Nevertheless, this emphasises that the advantages of using systematic surface support within the development mining cycle not only have an impact on the rockfall risk near the face but also to address some of the longer-term rockfall issues away from the face.

The issues relating to the group of rockfalls greater than two tonnes in Figure 12 are more likely to be linked with the ground reinforcement being less than adequate. This can be further sub-divided by causes or inadequacies such as corrosion, installation problems and design issues. Figure 12 shows ground reinforcement inadequacies with a breakdown by weight of rockfalls. All cases shown in Figure 12 were surface supported, therefore eliminating this as a cause of failure. Although the number of cases is limited, corrosion appears to be one of the prominent causes of ground reinforcement failure away from mining faces.

6 ROCKFALL FATALITIES

The consequences of rockfalls in the discussion above looked at injuries without mentioning fatal accidents. Detailed data on fatal accidents is difficult to obtain because of potential or pending litigation. This in itself is an issue as the lessons learned from such severe accidents can take a long time before they can be disseminated to the rest of the industry. Nevertheless, a small database of 23 fatal cases was assembled for this study. Many of the trends described above also apply to rockfall fatalities. For example, prior to 1998, many of the events occurred near the face in unsupported ground. Many of the cases involved relatively small rockfalls.

However, cases in recent years seem to break with this trend. Most recent rockfall fatalities were in supported ground, involving relatively large sized rocks (greater than 2 tonnes) that occurred more than 50 metres away from the active face. Therefore, the "lower risk" assessment given to areas away from the face due to the low exposure of personnel as discussed in section 4, needs to be qualified in terms of the severity or consequences of the few rockfall events that took place.

7 CONCLUSION

Ground support and reinforcement are the preferred control measures to mitigate rockfall risks. It is useful to sub-divide the risk associated with rockfalls according to their location from active mining faces. To reduce this risk, one could consider reducing the exposure of people to the face by modifying some of the mining cycle tasks or alternatively, attempt to reduce the occurrence of those small rockfalls by improved scaling techniques, or by using systematic surface support in mining faces and walls.

Rockfalls close to active faces have a higher risk of causing injuries as they occur in restricted areas where the activities are occurring on a near-continual basis. The large majority of these injuries are associated with the face, where rocks are either detaching from the face itself or from the backs within a half metre of the face. There is rarely any reinforcement or surface support installed in this zone unless ground conditions are

extremely poor (considered high hazard) or observed stress is high and support standards are increased in direct proportion to the hazard.

Away from the face, rockfalls also occur relatively frequently but the risk of injuries is lower. This is due to the low exposure of mine personnel to those rockfalls. Lack of surface support and corrosion of reinforcement elements are the main causes for these rockfalls. The consequences of any rockfalls can be very severe and it is noted that in recent years, most rockfall fatalities have occurred away from active mining faces.

The rockfall statistics collected for this study have greatly enhanced the current understanding of the risk of rockfalls in Australian underground metalliferous mines. Recent rockfall injuries occur near an active development mining face and originate from the face itself, the last half-meter of back near the face and from the walls. In "normal practices", these rock surfaces are rarely supported.

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Failure modes and support of coal roofs

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ABSTRACT: The ground stresses in coal roofs in underground excavations have been found to be substantially different from those in the surrounding stone. This may be related to shrinkage of the coal as it is dewatered and also drained of gas. The resulting stress field has the major principal stress being vertical and the principal horizontal stresses being substantially less than the vertical (as low as 40% and 20%). Over time the vertical stresses in massive coal or the onset of joint separations if coal cleat or joints are present. Roof support strategies and mine layout considerations need to be different from those adopted for stone roofs.

1 INTRODUCTION

Many Australian longwalls operate in thick seams (>5-6 m) and, depending on coal quality considerations, may leave coal in the roof on development. All these mines target very high production levels that come naturally from the exploitation of the thick seams. The mines have the same gateroad development demands that other longwall mines have, and most, if not all, have experienced roof falls and development shortfalls.

In many roadways with coal roof, falls of ground tend to happen close to the face (Figure 1), without warning in terms of noise, and often have steeply dipping structures associated with them (Figure 2). Large roof falls occasionally develop several years after mining. Conversely, flat coal roofs supported with spot bolts can be formed at depths of over 450 m, well beyond the expected depth for the onset of compressive failure of the coal.

These observations challenge some of the basic assumptions made by Australian coal mining geo-technical engineers. This paper presents a hypothesis for coal roof behaviour and discusses how the approach to mine planning and roof support design needs to be modified.

1.1 Roadway development methods

In Australia, continuous miners are used to form rectangular roadways that are typically 5.2 m wide and approximately 3.5 m high. The roadways are laid out in a grid comprising headings and cut-throughs.



Figure 1. Fall cavity formed close to the face in highly cleated coal.



Figure 2. Fall cavity bounded by joints.

To install the longwall faces, wider roadways are formed—say 8–9 m wide. Mining depths currently range from 80 m to 450 m.

Roof support is installed close to the face using roof bolters located on the continuous miners, or if placechange systems are used, 10–12 m long cuts outs are excavated by a remote-controlled miner and a separate bolting machine flitted into the place to install support. Roof support typically consists of bolts or dowels, roof straps, and mesh panels. Bolt length varies between 1.5 m and 2.7 m.

Seam dips are less than 10° , and mostly less than 5° . Continuous miners operate on a level floor, so the roofline is usually horizontal, with occasional 'shanty backs' forming naturally if a well-developed parting is present at the roofline.

1.2 Roof failure modes in coal measure strata

The rock mass in bedded strata such as coal measures typically comprises bedding partings between sedimentary units with bedding textures, and 2 sets of joints orthogonal to bedding with one set predominant. Bedding partings are typically smooth and planar and joints are typically rough and planar. This blocky structure is acted on by horizontal stresses, body stresses, and possibly vertical surcharges (Figure 3). In the vicinity of faults, additional discontinuities are often present—near normal faults there are discontinuities with dips as low as 50° – 60° , and near thrust faults there may be slickensided surfaces with dips as low as 10° .

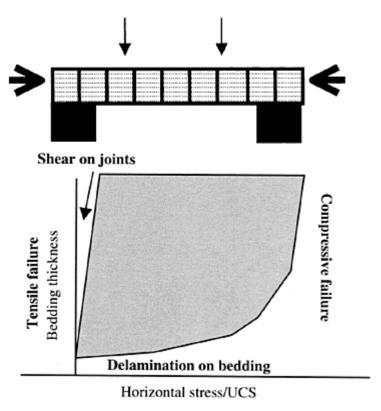


Figure 3. Failure modes in coal strata.

Design of roof support needs to recognize the range of failure modes in which this jointed bedded beam may fail (Figure 3). Failure modes can include:

- (a) Mobilisation of joints if there are low horizontal stresses and joints that define kinematically acceptable fall blocks. Low and possibly zero horizontal stresses have been inferred in the roofs of thick coal seams, under goaf edges in multiple seam mining, and possibly in tailgates with yielding pillars (Seedsman, 2003 & 2001). Joint distributions have not been well studied in coal measures, although in other sedimentary sequences it is known that joint spacing follows a negative exponential distribution (Priest and Hudson, 1976) and that spacing is related to bed thickness (Narr and Suppe, 1991).
- (b) Mobilisation of sub horizontal discontinuities if the spacing of the bedding partings is such that a buckling or snap-through failure of roof beams can occur under the influence of relatively low horizontal roof stresses. This type of failure can be analysed with voussoir beam methods (Sofianos and Kapensis, 1998).

For the case of coal with an unconfined compressive strength of 10 MPa, a density of 0.014 MNm^{-3} , and a modulus of 1.5 GPa, coal beams of 0.25 m– 0.3 m can support 1 m of coal surcharge when the horizontal stresses acting axially along the beam are less than about 3 MPa (Figure 4). As the imposed horizontal stress approaches the unconfined compressive strength (UCS) of the coal, the required beam thickness increases rapidly.

(c) Compressive failure of the rock substance if the horizontal stresses within the roof exceed the UCS of the material that forms the beam.

There is an additional failure mode case where roof joints dip at less than about 70°. This joint orientation

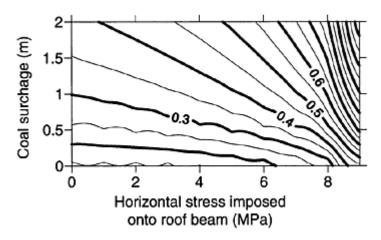


Figure 4. Contours of thickness of coal beam (in metres) to span 5.2 m wide roadways as a function of horizontal stress acting on beam and vertical surcharge.

allows shear slip to develop on the joints and the result is the formation of 2 cantilevers in the roof, which can then fail.

2 STRESSES IN COAL

A key starting point in understanding roof falls in coal is the realisation that horizontal stresses in coal seams may be substantially less than the vertical stresses. In Australian coal measures, the conventional wisdom is that the magnitude of the major principal horizontal stress is approximately twice the vertical stress, and that the magnitude of the minor principal horizontal stress is approximately 1.2–1.5 times the vertical stress. This remains the case for non-coal strata.

For coal, this general assumption should not be applied. If it were assumed that the horizontal stresses in coal measure sequences are related to tectonic events that applied sideways forces in a plane strain condition, lower horizontal stresses in coal would be anticipated. However, observations of coal roof failure modes and the results of overcore stress testing indicate that other mechanisms may be at play.

2.1 Testing from the surface

Coal bed methane programs over the last 10 years have routinely measured the minor stress in coal seams by pressurising borehole intervals in step rate tests. Stress data have been collected from NSW and Queensland coal seams, over a range of seams thickness and depths. Enever et al (2000) presents data that show that the minor stress is consistently about 40%-60% of the presumed vertical (Figure 5) and infer that the minor stress is a principal horizontal stress.

2.2 Overcoring underground

Stress measurements using the overcoring method have been conducted in coal seams (Seedsman et al,

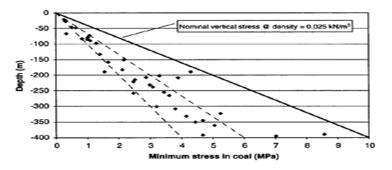


Figure 5. Summary of the minimum stress in coal seams (after Enever et al, 2000).

2002, Shen et al, 2003). Such tests can only be conducted in massive coals with poorly developed cleating and with widely spaced joints. After reviewing the published data and a number of unpublished stress measurements it is assessed that, in the absence of site measurements, the presumed stress field in coal roofs should have the 2 horizontal principal stress as low as 20%–40% and 50%–80% of the vertical stress, with the vertical being 50% less than that usually estimated from the depth of cover.

2.3 Model for the development of low stress fields in coal

Low stress fields have been obtained in overcore tests from 3 separate coal seams in Australia, and it is considered that they truly represent the stresses that are present in drained coal. It is considered that similar stress fields may be present in highly cleated coals in which overcore measurements cannot be conducted.

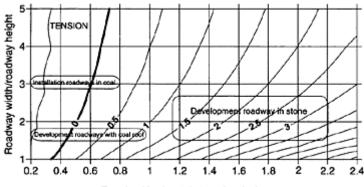
A characteristic of coal seams is that they are aquifers. As roadways are formed, the water pressure in the coal is reduced to zero around the openings. This means that the effective stresses in the coal increase and the coal must compress. For example, at 200 m depth of cover, a 6 m thick coal seam with a modulus of 1.5 GPa would compress by 8 mm on depressurising. Shrinkage of coal during drainage of gas is also well established (Dunn and Alehossein, 2002). It is suggested that as the coal shrinks away from the surrounding strata, there would be no shear stress transfer between coal and stone and a new stress field would develop. The way by which such a stress field would develop in coal is unknown. It is anticipated that a greater stress change may develop in highly cleated coals that have a lower modulus. It is noted that Dahlo et al (2003) discuss a similar mechanism to explain anomalous stress measurements in a hydro-electric project.

An implication of a shrinkage model is that with time the depressurisation of the coal may extend over a wide area such that the overburden cannot span over the void created by the shrinkage. The overburden could then settle down onto the coal and the vertical stresses could return to higher values associated with weight of the full overburden column—the horizontal stresses may not recover in the same way. If the vertical stresses do recover, then the ratio of the horizontal to vertical stresses may be even lower than the 20% ratio that has been suggested above.

2.4 Elastic stresses induced around roadways with coal roof

The way by which ground stresses are redistributed about a coalmine roadway is dominantly a function of the aspect ratio of the roadway and the horizontal to vertical stress ratio. A general indication of the stresses induced at the centreline of flat roof of a rectangular can be obtained using the closed-form solution for stresses about an elliptical/ovaloid excavation (Brady and Brown, 1985). Numerical methods should be used to obtain more accurate estimates for design purposes. Figure 6 presents the factor by which the magnitude of the far-field vertical stress should be increased or decreased in order to estimate the magnitude of the elastic horizontal stress at the roof centreline. Also shown in Figure 6 are typical aspect ratios for coal and stone roadways. Because of the elevated horizontal stresses in stone (say 2:1), the stresses in stone roof are compressive. If the coal had a stress regime directly related to the step test results (say 0.8:1), the

stresses in coal roofs would also be compressive. For coal roof exposed to a stress regime inferred from the overcore results, the roof stresses are weakly compressive to tensile,



Resolved horizontal stress/vertical stress

Figure 6. Factor to estimate the magnitude of the centerline horizontal roof stresses from the magnitude of the vertical stress.

and especially for the case of longwall installation roadways.

3 STRENGTH AND STRUCTURE OF COAL

Coal is a highly variable geological material. Geologically, coals are classified or 'ranked' in terms of their utilization as a fuel—low rank implies energy (steaming) coal and high rank implies metallurgical (coking) coal. Low rank coals tend to be dull and massive while high rank coals tend to be bright and highly cleated.

Coal seams can have bands of fine-grained sedimentary rock such as shale or claystones. The banding may be defined by bedding partings or there can be a transition between the various lithologies without discontinuities present. Where present, the spacing of the bedding discontinuities can vary from millimeters to tens of centimeters.

Coal is cleated and jointed. There are typically 2 cleat sets and 2 joint sets in coal, both typically aligned orthogonal to the bedding, with one set more dominant than the other. The author uses cleat to refer to the orthogonal discontinuity surfaces that are extremely closely spaced and with negligible persistence—these are probably related to the coalification processes. Cleats are more common in the high rank, bright coals. Lower rank coals tend to be less cleated, and there are examples of massive coking coals. Joints in coal are discontinuity surfaces that have persistences within the order of magnitude of the thickness of the coal seam. The spacing of joints may range from close to extremely wide. Near normal faults, joints in coal can dip in the range of say 60°–90°, parallel to the fault planes. Near thrusts, joints are closely spaced and coal presents as a highly friable material.

The issues of rank, banding, cleating, and jointing makes the selection of strength parameters for coal challenging. Assuming a Mohr Coulomb material, friction angles of 30° to 40° and unconfined compressive strengths ranging from about 10 MPa for coking coals to 40 MPa for thermal coals are obtained from 61 mm core samples. Tensile strengths of coal substance are often not measured—the author has found that a tensile strength within the range of 1/10 to 1/15 of the UCS is a reasonable approximation for laboratory-sized samples—say 1 MPa to 2 MPa.

Medhurst and Brown (1998) gives appropriate guidance on the selection of Hoek-Brown parameters for heavily jointed isotropic coal masses for a range of coals from bright coking coal to dull thermal coal.

4 ANALYSES

4.1 Failure modes for coal roofs above development roadways

From the discussions on bedding and jointing, it is apparent that the block sizes in coal can be of a similar scale to a roadway excavation. On this basis, it is appropriate to view coal as a blocky material and not a heavily jointed isotropic mass.

4.1.1 Compressive and tensile failure of coal substance

With the stress field that is developed in coal, it is highly unlikely that compressive failure of unstructured coal will develop in the centerline of roadways. By reference to Figure 6, the maximum induced horizontal stress in the roof for typical roadway dimensions will be about 0.5 times the vertical stress. Given the low vertical stress implied in the stress model, the horizontal stress will only exceed 10 MPa (an appropriate value for the unconfined compressive strength of coking coal) at depths greater than 1.5 km.

For the case of tensile failure of coal with a strength of 1 MPa, depths in excess of 250 m would be required. These simple calculations provide an explanation for the references to the observed flat coal roofs at depths of cover of up to 450 m.

4.1.2 Delamination

Delamination is always in possible roofs with banded coal even with only low horizontal stresses (Figure 4). The loading of the coal beams comes from self-weight and the surcharge applied by any overlying coal that is more banded and by overlying roof stone. In one mine that the author has studied the required coal beam thickness was approximately 0.5 m—this thickness being required to support 1.5 m of highly banded coal above. Operationally, the requirement was to leave 2 m of coal in the roof. When the mining horizon was raised, roof falls would rapidly develop once the massive coal at the base of the 2 m sequence was reduced in thickness to less than 0.5 m.

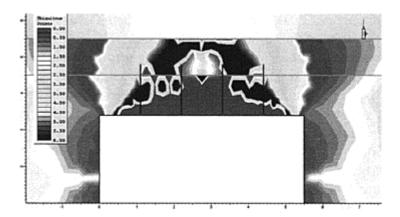


Figure 7. Phase² model of coal roof showing the lesser of coal and ubiquitous joint strength factor.

4.1.3 Joints

Figure 6 shows that many coal roofs can be exposed to tensile elastic stresses. Once the roof stresses go tensile, there is a possibility of roof collapses if the orientation of joints is such that kinematically acceptable blocks are formed—for instance if the roadway is aligned parallel to the strike of moderately spaced joints (Figure 2) or if the coal is closely jointed (Figure 1).

The onset of large zones of joint-controlled failure requires the horizontal/vertical stress ratio to be in the order of 0.2. A finite element analysis has been conducted (Figure 7) using a Mohr-Coulomb failure criterion for the coal with a tensile strength of 1 MPa, and vertical ubiquitous joints with a friction angle of 30° . There is a large failure zone near the roofline from which joint bonded blocks could fall. There is also a compressive stress arch developing above the zone of joint failure that would define the height of any fall. There are 4 bolts in the model and it is noted that the 2 centre bolts may not perform adequately as they are anchored in the potential failure zone.

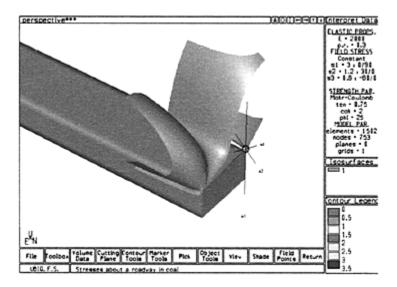


Figure 8. Isosurface for strength factor of 1.0 against failure of ubiquitous joints aligned parallel to roadway.

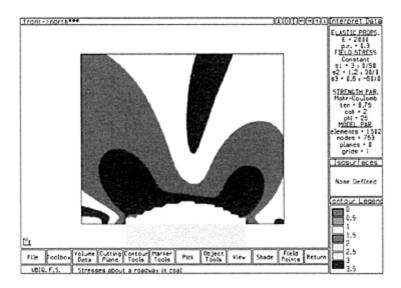
Care needs to be taken when interpreting joint failure modes underground. For example, if there was a single joint in the roof striking parallel to the roadway direction and offset to one side, the onset of tensile roof stresses could lead to the formation of a cantilever. Should this cantilever fail, the underground observer would notice a joint defining one side of the fall cavity and evidence of compressive stresses on the underside of the cantilever on the other side of the fall cavity.

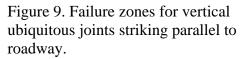
4.2 Bias in roadway deformation

Bias in roadway deformations to one side of the roadway or between headings and cutthroughs is often used as an indicator of elevated horizontal stresses. It is important to note that a similar bias is a characteristic of joint-bounded falls in coal.

Reference to Figure 6 readily shows that if the 2 horizontal principal stresses have significantly different magnitudes, a heading aligned parallel to the major horizontal principal stress could have tensile stresses induced in the roof, while the associated cutthrough which would be aligned parallel to the minor principal horizontal stress would have low compressive stresses.

A roadway aligned at an angle to the principal horizontal stresses may experience joint bounded falls biased to the side of the roadway which first intersects a line drawn parallel to the direction of the major principal horizontal stress (Figure 8 and Figure 9). In Figure 8 it can be seen that the biased failure zone (defined by strength factor of less than 1 for ubiquitous vertical joints aligned parallel to the roadway) is well developed at the face itself while on the other side of the roadway the failure zone is only just forming.





On a vertical plane taken 1 m behind the face (Figure 9), the strong bias to the left hand side of the roadway can also be seen.

5 IMPLICATIONS

5.1 Roof support design

The specification of coalmine roof support in Australia is dominantly the outcome of the observational method applied to mines with stone roof and high horizontal stresses. If such support patterns are applied to coal, there is little risk that the capacity of roof bolts to reinforce against a delamination mechanism would be exceeded. In fact, there is high probability that excessive support is being installed against a delamination mechanism, and there may be options to reduce support density or bolt capacity.

In low-stress, joint-controlled roofs, roof support can be based on dead-weight suspension. The weights involved are in the order of 150 kN/m of roadway advance, and well within the tensile capacity of rational bolting patterns. There is a concern that some of the support patterns derived from stone roofs may not locate bolt anchorage correctly. The most reliable anchorage will be found towards and above the sides of the roadway or in overlying stone. For roofs with multiple joints, the use of strap or mesh panels may be required to support joint-bounded blocks with dimension less than the bolt spacing.

5.2 Monitoring programs

It is important to recognize that roof falls require the coincidence of tensile stresses and sub-parallel joints. This coincidence may only happen rarely and as a result the hazard may be downgraded if the mechanics are not fully appreciated. As a simple example, consider a coal deposit with joint swarms over 4% of the area and a horizontal/vertical stress ratio of 0.5:1, with a mine layout that has cut-throughs and longwall installation roadways parallel to the joint swarms. If the cutthroughs were coincident with the joints, the roof stresses would still be compressive and no joint bounded falls would develop. The possible role of joints may be ignored and support plans not include a trigger to respond to joints. The wide installation roads are therefore exposed to a major hazard. The probability of a fall in a single installation road is equal to the probability of it being coincident with a joint swarm—a low 4%. On a life-of-mine basis with say 20 longwall panels, there is a 71% probability that one installation road will collapse if the possible role of joints is not recognised.

There is also a need to recognise that the pre-collapse movements of coal roofs undergoing joint bounded falls may not follow the same trends as those developed in delaminating stone roofs. Certainly, there may be less noise as there is less new rock breakage developing.

5.3 Mine planning

The primary consideration for mine layouts in thick coal seams is that they should be laid out oblique to the joint structure in the coal. Not only does this have the advantage of minimizing the number of kinematically acceptable roof falls, it also provides intrinsically more stable sides to the excavations. Once this is done, the next step should be to maximize the number of roadways aligned parallel to the minor principal horizontal stress so that the horizontal stresses acting across the roadway are maximized.

Massive coals should allow the formation of extended cuts and low roof support densities at depths in excess of 500 m. In this case, massive coals would be defined as those with banding more than say 300 mm–400 mm apart and not jointed or cleated.

6 CONCLUSIONS

Coal is a different material compared to the stone that encloses it. Its low modulus, water and gas bearing nature, and the range of discontinuities require consideration of possibly different stress fields and the onset of tensile stress and joint bounded falls. The behavior of coal roofs and the design of roof support needs to viewed in a unique framework and not as a continuum of the behavior of stone roofs.

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Rockfalls in Western Australian underground metalliferous mines

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ABSTRACT: This paper summarizes data on rockfalls in Western

Australian underground metalliferous mines from 1980 to 2003 as gathered by the Department of Industry and Resources. The data includes information on numbers of: underground employees, lost time injuries, fatalities and reported rockfalls. The data were analyzed on an annual basis to determine if any trends were present. The introduction of the Mines Safety and Inspection Act 1994 and Mines Safety and Inspection Regulations 1995 in December 1995, plus the MOSHAB Surface Rock Support for Underground Mines Code of Practice in February 1999 are discussed. Suggestions are made for the improvement of data collection. For comparison, data on the number of employees and rockfall lost time injuries in Queensland and New South Wales underground metalliferous mines are also presented. The limitations of accident and incident data are discussed. The use of positive performance measures is reviewed.

1 INTRODUCTION

The Department of Industry and Resources (DoIR) collects data on accidents and incidents in the Western Australian mining industry This paper presents the results of an analysis of the number of accidents and incidents involving rockfalls in Western Australian underground metalliferous mines from 1980 to 2003. The DoIR has several databases that were used to provide the data for these analyses.

Data held by DoIR was analyzed to determine if any trends were apparent. The following data from underground metalliferous mines were investigated:

- 1 Number of underground employees
- 2 Number of lost time injuries
- 3 Number of fatalities from all causes
- 4 Number of reported rockfalls

Some comparison with similar regulatory jurisdictions (Queensland and New South Wales) was attempted.

An earlier version of this paper was presented at the 2nd Biennial Workshop on Ground Control in Mines organized by The Chamber of Minerals and Energy of Western Australia Inc Ground Control Group (WA) on 20 June 2003.

2 DATA SOURCES

Four primary sources of data have been used for this analysis:

1 AXTAT database

2 Incident notification database

3 Information from the Queensland Department of Natural Resources and Mines 4 Information from the New South Wales Department of Mineral Resources

In addition, data on Western Australian underground metalliferous mining fatalities in McDermott et al. (1991) have been updated to cover the period from 1980 to 2002.

3 TERMINOLOGY

Some of the terms used in this paper are briefly discussed below:

AXTAT: "is a computerized system developed by the Western Australian Department of Industry and Resources for recording and retrieving information about disabling injuries resulting from accidents in the workplace", see AXTAT (2001). AXTAT was introduced on 1 January, 1987.

Accident: Caples (1998) described an accident as "a sequence of (unplanned) events that led to a bad outcome (injury, damage or loss)". The Macquarie Concise Dictionary (1988) defined accident as including: *noun* 1. an undesirable or unfortunate happening; casualty; mishap. 2. anything that happens unexpectedly, without design, or by chance.

Incident: Caples (1998) described an incident as "a sequence of unplanned events that could have led to a bad outcome, but didn't. These are often called 'near misses' or sometimes 'near hits'". The Macquarie Concise Dictionary (1988) defined incident as including: *noun* 1. an occurrence or event. 3. something that occurs casually in connection with something else.

Disabling injury: A work injury, not a lost time injury, that results in the injured person being unable to fully perform his or her ordinary occupation (regular job) any time after the day or shift on which the injury occurred, and where either alternative or light duties are performed.

Lost time injury (LTI): A work injury that results in an absence from work of at least one full day or shift any time after the day or shift on which the injury occurred.

Incidence rate: The number of injuries per 1,000 employees for a 12 month period, sometimes referred to as the lost time injury incidence rate (LTIIR).

Frequency rate: The number of injuries per million hours worked, also known as the lost time injury frequency rate (LTIFR). The incidence rate and the frequency rate are both measures of past events and as such are lagging indicators.

The incidence rate and frequency rate definitions above were taken from AXTAT (2004). It is noted that AS 1885.1–1990, also known as the *National Standard for workplace injury and disease recording*, has different definitions for these terms. As noted in AXTAT (2004) the National Standard includes a penalty of 220 workdays lost for each fatality, AXTAT keeps them separate with no penalty. AXTAT also calculates the incidence rate per thousand employees, while the National Standard calculates incidence rate per hundred employees.

Rockfall: The uncontrolled fall, detachment or ejection of rock, of any size, from the boundary of an excavation that causes, or has the potential to cause, injury, damage to equipment or damage to an excavation accessed by the workforce and thereby pose a hazard to the workforce or compromise the integrity of the mine structure.

Rockfall driving mechanisms include gravity, blast vibrations, seismicity, increased rock stress levels, decreased rock stress levels. Examples of rockfalls include: wedge failure, slab failure, rockbursts, strain-bursts, spalling, etc. Rockfalls are essentially driven by the forces in the rock mass—gravity and rock stress (pre-mining and induced).

Because of the ubiquitous presence of rock in the underground working environment it has the potential to present a range of hazards. Rocks removed from the boundary of an excavation by "controlled" mining processes such as scaling are not regarded as rockfalls. Rocks falling from pieces of equipment (e.g. loaders, trucks, etc.) would not be regarded as a "rockfall". Rocks falling down rises when disturbed from ladders, stages, pipework, etc are regarded as rockfalls. A run of ore or rock from a pass is not regarded as a rockfall. It is expected that a reasonable and practicable approach will be taken to classifying what constitutes a rockfall on a case by case basis. A more detailed description of the circumstances resulting in the rockfall, in the more unusual cases, would be useful.

In summary, if a "fall of ground" has the potential to pose a hazard to the health and safety of the workforce, no matter what its size or where it occurs, then it should be deemed to be a "rockfall". If in doubt, report it as a rockfall with an explanation of the incident.

Rock failure event: Failure of the rock mass at any location within a volume of the rock mass, not only at the boundary of an excavation. This term includes failure events that can occur at a distance from mining excavations. A more general term than rockfall.

4 STATUTORY AND REGULATORY ASPECTS

The Mines Safety and Inspection (MSI) Act 1994 and MSI Regulations 1995 came into operation on the 9 December 1995. Regulation 10.28, geotechnical considerations in underground mines, led to geotechnical issues being more widely recognized by industry. It is understood that this was the first time underground geotechnical issues had been explicitly recognized in Australian mining legislation.

Prior to 9 December 1995, the mining industry of Western Australia was regulated under the Mines Regulation Act 1946 and Mines Regulation Act Regulations 1976, which did not deal specifically with geotechnical considerations.

The Mines Occupational Safety and Health Advisory Board (MOSHAB) was established by section 90 of the MSI Act 1994. The MOSHAB (1997a) approved

Geotechnical considerations in underground mines guideline was issued in December 1997. The MOSHAB (1999) Surface Rock Support for Underground Mines Code of Practice (Code of Practice) was introduced in February 1999.

Thus, during the period being considered there were several developments that may have had a bearing on ground control in underground mines in Western Australia. Consequently, it was considered appropriate to investigate what trends, if any, there were in the number of fatalities and injuries caused by rockfalls plus the number of reported rockfalls in underground mines since 1995.

5 METHODOLOGY

The analyses were based on the annual numbers of underground employees and lost time injuries. A calendar year basis was used for some of the analyses because this best suited the implementation dates for the MSI Act 1994 and MSI Regulations 1995 in December 1995 and the MOSHAB (1999) Code of

Practice in February 1999. The numbers of lost time injuries were categorized by the type of accident, e.g. rockfall, overexertion, struck by object, etc. The more conventional approach based on LTIFR could not be used because the hours worked could not be allocated by type of accident. The data comparisons with two other States were done on a financial year basis, reflecting current reporting trends.

6 REQUIREMENT TO REPORT INJURIES AND INCIDENTS

AXTAT (2001) states that: "Sites which are designated as mining operations as defined by section 4 of the Mines Safety and Inspection Act (1994) are required to complete and submit to the Department an AXTAT Mining Injury Report Form whenever an injury occurs that prevents a worker from performing his or her ordinary duties."

This requirement is drawn from the MSI Act 1994:

"Notice of accident to be given

76.(1) Where a person suffers injury in an accident at a mine and is disabled by that accident from following his or her ordinary occupation, the manager must cause notice of the accident to be given—

- (a) in accordance with the regulations, to the district inspector for the region in which the mine is situated; and
- (b) if the injured person so requests, to the secretary or local representative of a trade union of which that person is a member.
- (2) The notice required to be given under subsection (1) must—
- (a) if the injury appears to be serious, be given by the fastest practicable method of communication as soon as it is reasonably practicable to do so, and must subsequently be confirmed in writing; and
- (b) if the injury appears not to be serious, be given in writing at the end of the month."

It is a requirement under the MSI Act 1994 to report any rockfalls in a mine to the district inspector in accordance with section 78(1), (2) and (3):

"Recording of occurrences in the record book

78.(1) The manager must immediately give notice to the district inspector for the region in which the mine is situated of an occurrence to which this section applies, whether or not any bodily injury to any person or damage to property has resulted from the occurrence, and must give to the district inspector such particulars in respect of the occurrence as the inspector may require.

(2) The manager must without delay record particulars of an occurrence to which this section applies in the record book.

(3) This section applies to an occurrence of—

(a) any extensive subsidence, settlement or fall of ground or any major collapse of any

part of the operations of a mine, or any earth movement caused by a seismic event; or (b) \dots "

The interpretation of section 78(3)(a) is likely to cause some discussion in the industry as to what reasonably constitutes a "rockfall". The author's interpretation of the adjective "extensive" is that it applies only to the word "subsidence" and not necessarily the remainder of the sentence. Our interpretation of "fall of ground" is that it refers to a rockfall that has the potential to pose a hazard to the health and safety of the workforce. This is consistent with the general obligation in section 9 of the MSI Act 1994 that requires, amongst other things, that "…so far as is practicable …employees are not exposed to hazards…".

It is a requirement under the MSI Act 1994 to report any potentially serious occurrences in a mine to the district inspector in accordance with section 79(1) and (2):

"Manager to report potentially serious occurrences

79.(1) The manager must inform the district inspector for the region in which the mine is situated of any occurrence at the mine which in the manager's opinion had the potential to cause serious injury or harm to health (other than an occurrence referred to in section 78) although no injury or harm in fact happened.

(2) The manager must inform the district inspector as required by subsection (1) as soon as practicable after the manager has ascertained the facts and circumstances of the occurrence and, if required by the district inspector, must provide a written report on that occurrence."

Implicit in section 79 of the MSI Act 1994 is the need to report 'near misses' or 'near hits' that pose a hazard to the workforce, including rockfalls, that were not reported under section 78(3)(a). The reporting of these potentially serious occurrences is consistent with MCA (2001), see later discussion on positive performance measures.

7 AXTAT DATABASE

The following data were obtained from the AXTAT database.

7.1 Number of underground employees

Figure 1 shows the number of underground metalliferous mining employees on an annual basis. Obviously, the more underground employees, the greater the likely exposure of people to a potentially hazardous working environment. There was a general increase in the

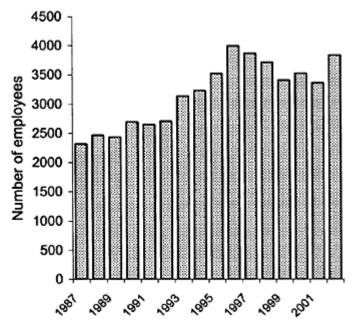


Figure 1. Number of underground employees from 1987 to 2002.

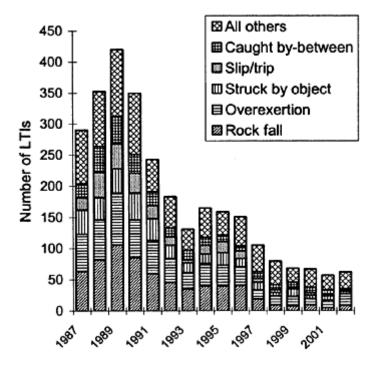


Figure 2. Number of lost time injuries 1987 to 2002 by type of accident.

number of underground employees from 1987 to 1996. The numbers then showed a generally decreasing trend from 1997 to 2001 before increasing in 2002.

7.2 Number of lost time injuries

The number of lost time injuries during the period 1987 to 2002 is shown in Figure 2 based on the type of accident. The accident types are the same as those appearing in the AXTAT annual reports. The data are shown as a stacked bar chart. Rockfalls are at the base of each bar and "all others" at the top, in the same order as the legend.

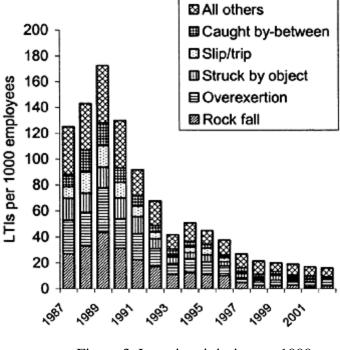


Figure 3. Lost time injuries per 1000 underground employees (incidence rate).

The data in Figure 2 are shown for five of the more common categories of lost time injury plus all others. The annual total number of lost time injuries peaked in 1989 and then generally showed a decreasing trend. Similarly, the numbers of rockfall related lost time injuries peaked in 1989 and then following a generally downward trend.

From 1987 to 1997 inclusive the annual number of lost time injuries caused by rockfalls was consistently the largest single category. From 1997 to 2002 rockfalls have been one of the top three largest categories of lost time injury.

From Figure 2 it can be seen that there has been an encouraging downward trend in the total numbers of lost time injuries. Since about 1999 there appears to be a tendency for the data to "level off".

To account for changes in the annual numbers of underground employees in the industry, the data in Figure 2 has been expressed per 1000 employees, see Figure 3.

Again, as evident in Figure 3, the downward trend in the number of lost time injuries per 1000 underground employees is encouraging. There appears to be a trend towards a "leveling off" of these data from about 1999, suggesting that continued improvements in performance are becoming increasingly more challenging to attain.

To better examine recent trends in Figure 3, the lost time injury data per 1000 employees from 1995 to 2002 is displayed in more detail in Figure 4. It will be

recognized that the data in Figures 3 and 4 are lost time injury incidence rate data. Also shown in Figure 4 is an indication of when the MSI Act 1994 and MSI

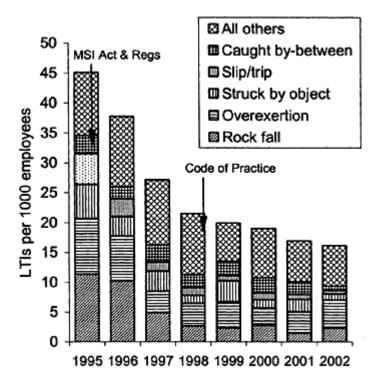


Figure 4. Lost time injuries per 1000 underground employees from 1995 to 2002 (incidence rate).

Regulations 1995 plus the MOSHAB (1999) Code of Practice were introduced.

As shown in Figure 4, there was a reduction in the total number of lost time injuries per 1000 employees from 1995 to 1998. However, the number of lost time injuries per 1000 employees due to rockfalls fluctuated during the five years from 1998 to 2002; the actual data were: 2.69, 2.35, 2.84, 1.49, 2.35 respectively.

7.3 Occupational health and safety initiatives

There were a range of initiatives directed at improving rock stability and occupational health and safety (OHS) in the industry during the period 1995 to 1999, including:

- 1 Mines Safety and Inspection Act 1994 and Mines Safety and Inspection Regulations 1995
- 2 WMC Resources Limited, Elimination of Fatalities Taskforce, rockfalls
- 3 MOSHAB (1997a), Geotechnical considerations in underground mines guideline

4 MOSHAB (1997b), Underground barring down and scaling guideline

5 DME, Development of High Headings Underground, High Impact Function Audit

6 DME, Geotechnical Considerations, High Impact Function Audit

7 MOSHAB (1997c), Inquiry into Fatalities in the Western Australian Mining Industry

8 MOSHAB (1998), Risk Taking Behaviour in the Western Australian Underground Mining Sector

Type of accident	Num ber	Perce ntage
Struck by (object or vehicle)	11	12.8
Electrocution	1	1.2
Explosive detonation	3	3.5
Fuming	2	2.3
Oxygen deficiency	2	2.3
Struck against	1	1.2
Caught by-between	9	10.5
Drowning (inundation)	6	7.0
Fall of person	11	12.8
Run of ore	6	7.0
Rockfall	34	39.5
Total	86	100.0

Table 1. Distribution of 86 underground fatalities by type of accident from 1980 to 2002.

9 MOSHAB (1999), Surface Rock Support for Underground Mines Code of Practice 10 MCA (1999), Safety Culture Survey Report of the Australian Minerals Industry

DME refers to the Department of Minerals and Energy, the name of the Department in 1996 and 1997 when these guidelines were developed.

It would be very difficult to separate the individual contributions made by these initiatives and other company work in the area of OHS. Consequently, it is not reasonable to draw any firm conclusions about the individual influence of the introduction of the MSI Act 1994 and the MSI Regulations 1995 or the Code of Practice on the lost time injury data presented above.

7.4 Number of fatalities

Table 1 presents the distribution of the 86 fatalities by type of accident that have occurred in underground metalliferous mines from 1980 to 2002. Table 1 is an update of Figure 11 from McDermott et al (1991) for underground metalliferous mines.

The data in Table 1 are the numbers of each fatality that occurred in each type of accident and the percentage of the total that this number represents. Thus, the number of fatalities resulting from rockfalls was 34, representing 39.5% of the total (86).

From Table 1 it can be seen that rockfalls are by far the single largest type of fatality in underground metalliferous mines. Rockfall fatalities were more than three times more numerous than the next largest fatality type (34 versus 11). This demonstrates that rockfall fatalities are a major issue to be addressed in the provision of a workplace where "...so far as is practicable...employees are not exposed to hazards...".

One interpretation of Table 1 is that it summarizes some of the principal hazards found in the underground mining environment over a 23 year period. The six most common types of accidents resulting in fatalities summarized in Table 1 account for approximately 90% of all fatalities. These were:

1 Rockfall

- 2 Struck by (object or vehicle)
- 3 Fall of person
- 4 Caught by or between
- 5 Drowning (inundation)

6 Run of ore

Fatalities associated with uncontrolled movement of rock and people (ie rockfall plus fall of person) account for more than half of all fatalities.

Data from McDermott et al (1991) showed the distribution of the 54 fatalities that had occurred in underground metalliferous mines from 1980 to 1991. During this 12 year period there were 19 rockfall fatalities in underground metalliferous mines. As a percentage of the total number of fatalities this represents 35.2%. (Note: One rockfall fatality in an underground coal mine was excluded from the data used in this analysis.)

During the 11 year period from 1992 to 2002 there were a total of 32 fatalities in underground metalliferous mines of which 15 were rockfall fatalities, 46.9% of the total. This is an increase in the percentage of fatalities that resulted from rockfalls over similar time periods, see Table 2.

7.5 Hazard identification

A key question for any underground mine would be: "What are the principal hazards that pose a serious risk to the health and safety of our employees?" The answer to this question is likely to be different at each mine site. As a starting point, it may be useful to review previous industry experience to determine the most frequent types of accident that have resulted in fatalities. The potential hazards at a particular mine would then need to be identified and evaluated against this industry experience. Such an approach is necessarily a retrospective or reactive analysis of past industry experience and only provides a partial answer to the above question.

Possibly more important will be the identification of the principal hazards that are unique to a particular mine site. These identified hazards need to be assessed,

Period (years)	Rockfall fatalities number (percentage)	Total fatalities number
1980–1991 (12)	19 (35.2%)	54
1992–2002 (11)	15 (46.9%)	32
1980–2002 (23)	34 (39.5%)	86

Table 2. Comparison of rockfall fatality data.

ranked and treated in an appropriate manner. A forward looking or proactive approach to hazard identification is essential to sound risk management.

8 INCIDENT NOTIFICATION DATABASE

The DoIR incident notification database contains, amongst other things, reports of rockfalls. This database has complete data on an annual basis from 1995 to 2002.

The incident reports can be sorted a number of ways, including: (1) by type, (2) by injury occurrence (ie fatality, injury, no injury) and (3) either surface or underground. This approach was used in this analysis. This analysis is based on the following assumptions:

- 1 All the rockfalls that occurred in underground mines were reported in accordance with section 78 of the MSI Act 1994
- 2 All rockfalls were uncontrolled falls of rock and were not caused by scaling or as a result of localized blasting activity

The localized blasting activity referred to above includes "pop" or "stab" holes drilled into large potentially unstable blocks, charged with explosives and then fired in an attempt to bring down the block. A rockfall after production or development blasting activities, when the workforce had re-entered the mine and were potentially exposed to the hazard, would be classified as a "rockfall".

These data represent the reported incidence of rockfalls. There is, by definition, no rational means available for determining the total number of rockfalls (reported and unreported).

8.1 Number of reported rockfalls

Figure 5 is a stacked bar chart showing the annual total number of reported underground rockfalls on the basis of injury occurrence from 1995 to 2002. The data represented in Figure 5 consists of a total of 644 reported rockfalls, made up of 499 no injury reports, 134 injury reports and 11 fatality reports.

There was an increase of 89% (55 to 104 reports) in the annual total number of reported rockfalls from 1996 to 1998. During the period of 1998 to 2000 the annual total number of reported rockfalls was fairly similar at about 100 per year. In 2001 the total

number of reported rockfalls decreased by 44% from 101 to 57 reports. The total number of reported rockfalls increased in 2002 by 74% from 57 to 99 reports.

From Figure 5 it can be seen that there was a considerable increase in the number of reports involving no injury during the period 1995 to 1998. During 1998 to 2000 there was a similar number of reports involving no injury, a decrease in 2001 followed by an increase in 2002.

The number of reported injuries from 1995 to 2002 has shown some variation. There was a decrease in reports from 1995 (24) to 1998 (16), an increase in 1999 (18) followed by a downward trend in 2000 (14), 2001 (10) and an increase in 2002 (15). The overall downward trend in reported injuries is commendable. A "leveling off" in these data, albeit variable, during recent years is noted. This is similar to previous trends noted in the numbers of lost time injuries per 1000 employees, see Figure 3.

Recently the number of reported rockfall fatalities has been considerably lower than it was during 1996 (four) and 1997 (four). One rockfall fatality was recorded in each of 1995, 1999 and 2000.

During the eight year period there were 134 injury reports in a total of 644 reported rockfalls. This suggests that the likelihood of an injury from a reported rockfall was approximately 1 in 5. Similarly, the likelihood of a fatality from a reported rockfall was approximately 1 in 60. This highlights the need for the unbiased reporting of all rockfalls including those that involved no injury.

It would be prudent to recognize that no injury rockfalls present learning opportunities in terms of the adequacy of the ground control systems at the mine. The no injury data in Figure 5 can be considered to represent failures of the ground control systems at the various mines. This suggests that reliability and risk assessment concepts may present alternative methods of analyzing ground control system performance.

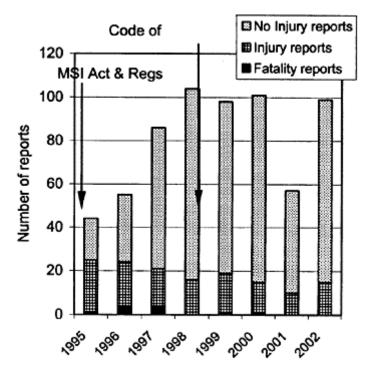


Figure 5. Annual number of reported rockfalls from 1995 to 2002.

9 SUGGESTIONS FOR IMPROVEMENT OF DATA COLLECTION

The following suggestions are made for the Western Australian mining industry to ensure the completeness of data in the AXTAT database:

- 1 Complete all the relevant fields in the Injury Report form. In particular, the location of the incident event in the mine should be described as fully as possible.
- 2 AXTAT information is required two weeks after the month being reported on. The prompt reporting of incidents and injuries will facilitate timely data analysis.
- 3 The legibility of Occurrence Forms can create a challenge to arrive at the correct interpretation. Data submission is presently being reviewed with the eventual aim of introducing the electronic submission of forms. Data integrity issues will need to be discussed and satisfactorily resolved.

10 COMPARISON WITH QUEENSLAND AND NEW SOUTH WALES DATA

The Queensland Department of Natural Resources and Mines and the New South Wales Department of Mineral Resources have provided data on the numbers of underground metalliferous employees and the rockfall lost time injuries.

The numbers of underground metalliferous mining employees in Western Australia (WA), Queensland (QLD) and New South Wales (NSW) from 1997/98 to 2002/03 are shown in Figure 6.

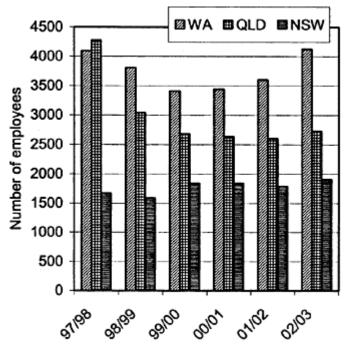
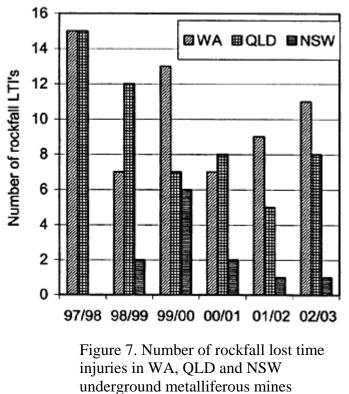


Figure 6. Underground metalliferous mining employee numbers in WA, QLD and NSW.



(includes fatalities).

From Figure 6 it can be seen that the number of underground metalliferous mining employees in Western Australia and Queensland both fell in the first three years. The Queensland employee numbers leveled off while those in Western Australia showed an increasing trend. In New South Wales the employee numbers have been fairly consistent during the past six years.

The numbers of lost time injuries due to rockfalls in Western Australia, Queensland and New South Wales underground metalliferous mines from 1997/98 to 2002/03 are shown in Figure 7.

From Figure 7 it can be seen that the number of rockfall lost time injuries in Western Australia has fluctuated during this six year period. There appears to be a general downward trend. It is noted that for the last three years there appears to have been an increasing trend in the number of rockfall lost time injuries in Western Australia. The Queensland data has shown a general downward trend with some fluctuations during this period. The New South Wales rockfall lost time injury data appear to have been consistently lower than Queensland and Western Australia during this six year period. The New South Wales lost time injury data for 1997/98 were not available.

In Figure 7, the lost time injury data for each State included the number of rockfall fatalities. The rockfall fatalities were: Western Australia 1997/98 (four), 1998/99 (one), 2000/01 (one); Queensland 2001/02 (one) and New South Wales 1999/00 (four).

It was not possible to express these data on a lost time injury rate per 1000 employees because some of the employee numbers data were unavailable.

Fluctuations in the numbers of rockfall lost time injuries can be observed in Figure 7 during the past six years. A sustained reduction in these numbers is a challenge for the mining industry. Alternative measures of occupational health and safety management performance may be necessary to bring about the continued reduction in rockfall lost time injuries.

11 LIMITATIONS OF ACCIDENT AND INCIDENT DATA

It is recognized that the collection of data on accidents and incidents is essentially a summary of past experience, a history of what has occurred. LTIIR and LTIFR are lagging indicators of OHS performance that measure what has happened in the past. It is useful to document past experience and preferably, learn from that experience. Such an approach is of limited effectiveness as it is primarily looking backwards and is essentially reactive in responding to past events. A more positive approach would be to look forwards, as well as backwards, and be proactive where possible. The essence of risk management is a requirement to be proactive rather than reactive.

Some of the limitations of accident and incident data collection have been recognized by a number of authors including Amis and Booth (1992) and more recently Shaw (1998), CMEWA (1998) and MCA (2001). The following seven points were taken from Appendix 1 of Shaw (1998). Most of these points are directly relevant to the issue of rockfall accidents and incidents. For the sake of completeness all seven points have been included. A brief discussion of each point from a rock stability perspective has been provided by the authors.

"Relying on incident data as the sole approach to measuring OHS performance has a number of problems:

1 Incident data are particularly limited for measuring the effectiveness of control of core risks. Core risks lead to high consequence, low probability incidents. The absence of an unlikely event is not, in itself, proof that the core risk is effectively controlled. The hazards which create core risks are not always the same as those which cause less serious but more frequent lost time injuries."

The issue of core risks is directly relevant to rockfall hazards. From Figures 2 and 3 it can be seen that the number of lost time injuries due to rockfalls is broadly comparable with several other types of lost time injury. However, Table 1 demonstrates that fatalities caused by rockfalls are very much greater in proportion to other types of fatalities. Consequently, it may not be reasonable to assume that low LTI or LTIIR numbers for rockfalls necessarily means that the core risk of fatalities caused by rockfalls is being adequately controlled.

2 "Incident data measures failure, not success. An accident or incident is evidence of a failure of OHS management. It does not tell an enterprise about aspects of its OHS management system which are working successfully Without information about what is working well, it is difficult to build on the system's strengths. Incident data only allows reaction to failure not proactive control of risks."

This is applicable to rockfall accident and incident data. A mine may have a soundly based ground control program. The success of this program would be more difficult to measure by lagging indicators such as LTI. Other more appropriate measures may include: number of reported rockfalls in various areas of the mine on say a monthly basis; number of geotechnical inspections conducted on a monthly basis, planned versus actual; the percentage of employees trained in ground control; the cost of remedial work to re-support areas that have experienced rockfalls, etc.

3 "Incident data will fluctuate at random. Especially in small enterprises or with a small number of accidents and incidents, changes in incident rates will not be statistically significant. This means that changes in incident rates may not be because of anything an enterprise is doing right (or wrong) in management of OHS. The changes might simply be a result of the expected fluctuation within a range of what is normal. This means that incident data is of limited use to support accountability of managers or supervisors, such as in performance appraisal, because changes in accident rates are not always a direct result of what a supervisor does."

This is applicable to rockfall accident and incident data. For example, local ground conditions will vary cut by cut in a development heading. This is simply an expression of the natural anisotropy and heterogeneity found in a rock mass. A deterioration in the ground conditions may be due to this inherent variability of the rock mass and is obviously not under human control. Deteriorating ground conditions may result in the potential for more rockfalls. Thus, the occurrence of more rockfalls may not be due to anything that the workforce, supervisors or management are doing. Rather, the inherent variability in the ground conditions becomes another factor that needs to be recognized and responded to in a timely manner.

4 "Incident data are like waiting until the end of the game to see how you went, not keeping track as the game progresses. Incident data reflect the success, or otherwise, of safety measures taken some time ago. Many types of occupational injury and illness are a result of exposures to risk which have occurred many years ago or over many years.... In any workplace, interventions must be able to be evaluated as you go—to fine tune, to identify and address confounding factors and to build preparedness to try a new method of working."

Again, it would be very useful to have more immediate measures of performance to determine if the corrective action has been adequate. This is particularly relevant to ground control, rock support and reinforcement and geotechnical issues generally. Each mine is encouraged to develop their own measures of performance that are relevant to their mining environment.

5 "Incident data do not measure the incidence of occupational diseases where there is a prolonged latent period. Occupational diseases are a major risk in the mining industry."

This is not likely to be an issue with rockfalls as the effects are much more immediate.

6 "Incident data measure injury frequency and severity, not necessarily the potential seriousness of the incident. For example, a stubbed toe gets measured, but someone

just missing being hit by a falling rock doesn't count, even though it's a sign of a much more serious OHS risk."

This is directly relevant to rockfall hazards. Rockfalls can potentially be very serious incidents, especially in areas where there is a high probability of workforce exposure to rockfalls. This simply confirms the importance of reporting all rockfalls because of their very high potential to result in a lost time injury or fatality.

As previously noted, data in Table 1 demonstrated the predominance of rockfall fatalities as a proportion of all fatalities. In general terms, Figure 5 demonstrates that there were approximately four reported rockfalls for every reported injury from 1998 to 2002 inclusive. This suggests that there was approximately a 1 in 5 chance of being injured in a reported rockfall incident.

7 "Incident data conceal the range of other influences on outcome measures. Many features of the workplace, industry and economy influence incident rates. As well as the enterprise's OHS management system itself, incident data can be influenced by different definitions of 'incident' or 'accident'. What is a lost time accident in one enterprise doesn't even get a mention in another. Different return to work policies and procedures can result in the same injury or incident being measured differently. Furthermore, if a lost time injury is defined as missing one shift, even the day of the week or time of the day on which the incident occurred can affect the measurement. A twisted ankle on Monday morning may require an absence until Wednesday and so count as a lost time injury. The same injury on Friday afternoon may see the injured worker back on Monday and thus not affect the incident issue. This means that comparing incident rates is not always comparing the effectiveness of the OHS management systems-you may not be comparing 'apples with apples'. Other influences include the worker's compensation system, other organizational changes, perceptions of job security or previous injury experience. Relying on incident data alone does not allow examination of the range of influences to determine the effect of OHS management initiatives as opposed to the effect of other changes inside or outside the enterprise or even industry."

This is applicable to accident and incident data generally, not just to those involving rockfalls.

In summary, accident and incident data have served the industry well as demonstrated by the improvement in OHS performance shown in Figures 2 and 3. Recent experience suggests that continued improvement in OHS performance, as measured by lagging indicators such as LTIIR and LTIFR, will be more challenging. It would be preferable to develop leading indicators of OHS and ground control performance that are more directly related to the identified hazards at each mine site. This would facilitate monitoring of the mine site specific processes that have been put in place to control the identified hazards. Some potential indicators are briefly discussed below.

12 POSITIVE PERFORMANCE MEASURES

A number of organizations, including the Minerals Council of Australia (MCA), have recognized the limitations of accident and incident data. MCA (2001) presents an informative discussion of "positive performance measures" that would appear to be more appropriate leading indicators of OHS performance.

MCA (2001) states at page 1: "A positive performance measure (PPM) is a measure of a proactive leading activity necessary to control loss and damage. It is an upstream process measure rather than a downstream outcome measure."

MCA (2001) proposes a generalized process model for company operations consisting of three stages:

1 Inputs (people, machinery, raw materials (rock)

2 Process (the mining or treatment process being used)

3 Output (product, sub-standard product)

Table 1 of MCA (2001) presents examples of positive performance measures that have been grouped into the three stages.

Table 3 provides a range of suggested PPMs that may be appropriate for some mines. The need to develop alternative PPMs will be driven by the identified hazards unique to a particular mine site. Each site would have the flexibility to develop PPMs that were appropriate for that site.

The necessity for individual PPMs would change over time as particular areas of concern were brought under control. The focus could shift to other areas of concern where new PPMs may need to be developed.

It is noted that "falls, unstable ground incidents" are included in Table 1 of MCA (2001), a clear recognition of the importance of reporting rockfalls. A rockfall PPM could, for example, be stated in terms of the number of reported rockfalls in the main decline, workplaces, oredrives, etc per month. A decreasing trend in this PPM (i.e. less reported rockfalls) with

Activity measures ¹ (Inputs)	Focus areas ² (Processes)	Action plans ³ (KPIs) (Outputs)
Task observations, conducted/scheduled Scheduled safety meetings	Factors producing strain/sprain incidents Exposures exceeding standard—noise/dust	% Supervisors trained in OHS % Managers attend leadership in OHS
Incidents investigated/close out within x days	Explosive incidents/near misses	% Managers conducting SMATs/Audits
Risk assessments completed	Fires/ignitions on equipment	% Risk assessments on major risks
JHAs conducted	Isolation devices	% Injured employees rehabilitated
SWPs reviewed, quota	Falls/unstable ground incidents	% Corrective actions outstanding

Table 3. Positive performance indicators examples (after Table 1 of MCA (2001)).

Inspections, % completed/scheduled	Hazard identification training—% completed	% Equipment retrofitted—fire systems, positive isolators
Audits completed % completed/scheduled	% Employees participating in wellness sessions	% Supervisors with First Aid certificates
Emergency exercises conducted/scheduled	% Employees failing fitness for work tests	
Results of audits-ratings	% Chemicals assessed on site	

1. Focussed on safety commitment and effort in safety management.

2. Focussed on upstream aspects of processes in main areas of risk.

3. Focussed on business/safety plans for operation.

time may be indicative of improved ground control measures and/or improving ground conditions. However, an increasing trend with more reported rockfalls may be a cause for concern. It is encouraging to see the collection and analysis of data on reported rockfalls being recognized as an important issue for the mining industry.

13 DISCUSSION

The preceding analysis highlights the serious hazard posed by rockfalls to the health and safety of the workforce in underground metalliferous mines. The following discussions are presented to raise important issues and should not be construed as definitive recommendations. Further work will be required to develop our understanding of these important issues.

There is a need to have reliable and objective data on the number of reported rockfalls of all types (incidents and accidents). This is particularly so for reported rockfall incidents that, by definition, involve no injury. The under-reporting of rockfall incidents, should this occur, only serves to bias the data and make the likelihood of rockfall injury (accidents) appear worse than it may be.

There is a need to set performance criteria for ground control systems so that their adequacy can be reviewed on a regular basis. The ground control system could be considered to consist of: drilling and blasting, scaling, assessment of ground conditions, and, if applicable, ground support hardware. The adequacy of the ground control system should be evaluated in terms of its objective, ie the ability to minimize, so far as practicable, the frequency of rockfall incidents and accidents. Current measures of system adequacy appear to focus more on the amount and/or type of ground support rather than the objective to be achieved.

Ground support design approaches generally consider: the need for ground support; and, if required, the number, spacing and length of rockbolts plus the possible use of surface restraint (e.g. mesh, shotcrete, etc). The support design criteria should include: expected load (or demand) to which the support will be subject and load capacity of the support.

The ground control system should include some measure of performance over time and space. These performance measures may include: number of rockfall incidents per unit of time (e.g. week, month, etc) per kilometer of development; mean time between rockfall incidents (e.g. one incident every six months, etc); number of rockfall incidents in particular areas of the mine (e.g. access decline, subsidiary declines/inclines, intersections, oredrives, drawpoints, permanent installations, etc). It may be useful to combine some of these in terms of location, time and space, e.g. number of rockfall incidents in the main decline per month. The length of development being considered must be sufficiently large to make the criteria meaningful. There would be little point in measuring the number of rockfall incidents per, say, metre of development.

Such an approach would be a useful first step in the development of positive performance measures that more directly monitor upstream process activity and thus provide more rapid feedback on the adequacy of control measures being taken. These performance measures should be regularly updated and discussed with the workforce to develop a shared understanding of the issues involved. As previously noted, these measures will not remain static but evolve with time as required.

The mining industry has the opportunity to critically review current methods of measuring OHS performance. While statistics based on accident and incident data have served the industry well in the past, it is now appropriate to move forward to more proactive positive performance measures. Consequently, there is a need for on-going work, on an industry wide basis, to develop more appropriate measures of hazard identification, incident occurrence and system performance not only in the area of ground control but also more generally.

14 CONCLUSIONS

The following conclusions are drawn from the above analysis:

- 1 The number of underground employees in Western Australia has increased from about 3400 in 1999 to 3800 in 2002.
- 2 The total number of lost time injuries has shown an encouraging downward trend since 1989. During the past four years this trend has shown a tendency to level off. This may be an indication of the difficulty in achieving meaningful performance improvement on a continuing basis.
- 3 The total number of lost time injuries per 1000 underground employees also demonstrated a commendable downward trend since 1989.
- 4 The annual number of rockfall lost time injuries were comparable with other types of accident resulting in lost time injury.
- 5 From 1980 to 1991 (12 years) rockfall fatalities accounted for 35.2% of all fatalities. Similarly, from 1992 to 2002 (11 years) rockfall fatalities were 46.9% of all fatalities.
- 6 During the period from 1980 to 2002 the number of fatalities caused by rockfalls was more than three times greater than the next most common cause of fatality.
- 7 The comparability of numbers of each type of accident in lost time injury data is in contrast with the much larger proportion of fatalities due to rockfalls. This demonstrates the difficulty of using accident or incident data as the sole measure of the effectiveness of controlling core risks.
- 8 The total number of reported rockfalls increased from 1995 to 1998, remained fairly similar in 1998, 1999 and 2000, decreased in 2001 and increased in 2002.

- 9 From 1995 to 2002 the likelihood of an injury occurring in a reported rockfall was approximately 1 in 5.
- 10 The numbers of underground metalliferous mining employees in Western Australia was higher than Queensland and New South Wales since 1999/00.
- 11 There have been fluctuations in the number of rockfall related lost time injuries in Western Australia since 1997/98. An overall downward trend is present from 1997/98 to 2002/03. However, the data from 2000/01 to 2002/03 have shown a upward trend.
- 12 Queensland rockfall lost time injury data have shown a general downward trend from 1997/98 to 2002/03.
- 13 The New South Wales rockfall related lost time injury data appear to be lower than Queensland and Western Australia from 1998/99 to 2002/03.
- 14 It may not be reasonable to assume that low LTI or LTIIR numbers for rockfalls necessarily means that the core risk of fatalities caused by rockfalls is being adequately controlled.
- 15 Alternative OHS performance measures need to be developed to measure the proactive work done to control identified rockfall hazards.
- 16 One way forward may be the adoption of "positive performance measures" (PPM) that focus on proactive process measures rather than solely on measures of historical performance.
- 17 The use of PPMs such as "falls, unstable ground incidents" is a positive step.
- 18 The systematic collection, recording, analysis and communication of data on reported rockfalls, whether resulting in injury or not, is believed to be an important approach for the industry to develop further.

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The views expressed in this paper are those of the authors and do not necessarily reflect the policy of the Department of Industry and Resources.

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Back analysis of block falls in underground panel cave excavations: The experience in panel caving at El Teniente Mine-Codelco Chile

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ABSTRACT: In this paper are introduced four cases of block failure from Reservas Norte Sector, at El Teniente Mine. Block stability analysis are carried out using specialized software packages, with the purpose of obtaining geometrical characterization of key-blocks. An hypothesis of failure controlled principally by an unfavorable induced stress ratio is introduced, and a sensibility analysis is carried out—taking the geometric information of fallen blocks—to draft the induced stress ratio that governs key-block fall. Finally, one of the case studies is analyzed with more detail to find out condition of stresses (stress ratio and stress magnitude) that determine wedge failure.

1 INTRODUCTION

The El Teniente Mine is a Codelco-Chile underground copper mine. It is located in the first elevations of the Andes in the central zone of Chile (South America), about 70 km SSE from the capital city, Santiago.

The El Teniente porphyry copper orebody is one of the largest known copper deposit in the world. It includes andesite, diorite and hydrothermal breccias of the Miocene era as the main lithologies.

The main structural feature of the orebody is a stock-work of multi-directional veins and veinlets. The veins are principally cemented with anhydrite, quartz and sulphides.

A chimney of subvolcanic breccias known as the "Braden Pipe" postadates the copper-molybdenum mineralisation. It has an inverted cone shape and the hydrothermal mineralisation is distributed around this pipe over a variable radial extension of 400 m to 800 m, with mineralogical associations of variable strength.

The mineralisation has two very different forms, the secondary ore is located near the surface and the primary mineralisation is at greater depth.

The primary ore can be described as a high cohesion and impermeable rock mass. The stockwork veins, containing the original mineralogy, are sealed. According to a geomechanical behavior, the primary rockmass could exhibit brittle, often violent failure under a high stress condition.

2 EXPLOITATION OVERVIEW

El Teniente Mine began operations in 1906. Since then, various exploitation methods have been used in productive sectors located in secondary mineral. The methods range from "raised work over mineral" combined with shrinkage stoping and pillar recovery to block caving.

Later, as a consequence of deeper productive sectors and changes in the physicalmechanical properties of the rock, the exploitation of primary ore (lower grade, stiffer, harder and with coarser fragmentation than the secondary ore) has resulted in the mechanization of mining operations. This situation required a change from the standard block-caving method used in secondary ore (primarily characterized by manual or semimechanized ore transfer) to the panel-caving method, in which fully mechanized ore transfer is continuously incorporated into the production area—i.e., a dynamic caving face.

Knowledge gained over the years concerning primary ore exploitation with conventional panel caving (200 million tons extracted to date) has indicated that the advance of the caving face is the main cause of gallery damage in levels below the UCL. Experience has also shown that a variation of conventional panel caving, the "preundercut," reduces the degree of gallery damage in the levels below the UCL, as well as the possibility of rockbursts associated with the advance of the undercut face. Preundercutting basically consists of advancing the undercut ahead of all development in the lower levels. All production-level development is made behind the cave front and under the caved area.

3 MECHANISM OF BLOCK FAILURE

In jointed rock masses the different geological arrangement form blocks or wedges that could be unstable, with respect to the position of the underground excavations, under different stress conditions in the context of a specific mine. These critical blocks are usually mentioned as key-blocks. Unless steps are taken to support these loose wedges, the stability of the opening may deteriorate rapidly. If the excavation has been supported, the block movement tendency will transfer loads to the support system, which could fail if they have not designed to handle these loads.

Naturally the form of any such defined block has an obvious impact on the stability of the excavation. In agreement with the observations by Windsor, Kuszmaul and Mauldon it can be considered that tethraedral blocks are the most common type. Their base, area, apex length and volume define blocks. In order to assess the global stability of an individual key-block it may be used the limit equilibrium method, after Hoek and Brown and the influence of reinforcement, after Li.

The mechanical (geotechnical) properties of the rock mass (considering both the properties of the structure fillings and the intact rock) are very important issues in block stability mechanism, therefore the level of knowledge about them establish the grade of belief in the estimations. The properties of the discontinuities can be defined by the Mohr-Coulomb criterion.

4 ESTABLISHING AN HYPOTHESIS OF BLOCK FAILURE

The most dominant factors influencing block failure are:

- Geometrical relationship between joints and excavation roof and walls.
- Joint characteristics (low cohesive and frictional strength).
- In situ or mining induced field of stresses (stress ratio and stress magnitude).

According to field observations at the mine, many blocks that have been stable during a long time fall because a change in the condition of field stresses occurs. Therefore, hypothesis of failure controlled principally by an unfavorable induced stress ratio is introduced in this paper, considering the following steps of key-blocks stability analysis after the hypothesis is established:

- Identifying key-block and acquisition of data from the field.
- Analyzing block stability using key-block software.
- Developing of sensibility analysis.
- Studying one of the case histories.

The different assumptions are introduced in the development of each step.

4.1 Identifying key-block and acquisition of data

After block failure occurs, a geological work at field is developed, identifying geotechnical (mechanical) properties of weakness planes (joints) and also identification of installed reinforcement and damage of the excavation.

4.2 Analyzing stability using key-block software

In this paper are introduced four cases of block failure from Reservas Norte Sector. Block stability analysis are carried out with PT-Block Tunnel Software (Pan Technica Corporation) or Unwedge 2.2 (Rocscience 1991), with the purpose of obtaining geometrical characterization of key-blocks. General assumptions for both programs are the following:

- All the joint surfaces are planar.
- Joint surfaces are continuous enough to extend entirely through the volume of interest.
- Failure by cracking of blocks or joint extension is not considered.

- Blocks defined by the system of joint faces are assumed to be rigid; displacements are by sliding only.
- Stability is determined by the limit of equilibrium where driving forces are compared to resisting forces.
- The analysis is based the assumption that wedges are subjected to gravitational loading only. Therefore the stress field in the rock surrounding the excavation is not taken into account.
- Unwedge calculates the maximum sized wedges which can form around the excavation, with a maximum of three structural planes analyzed at one time.
- In the case of PT-Block Tunnel Software strength of joints is giving by sliding friction angle. Tensile and cohesive strengths are always zero.

Table 1. Input information of joints at intersection of Calle 7 and Zanja 30, production level of Reservas Norte.

Joint	Dip (°)	DipDir (°)	Comments
J1	89°	346°	Open joint
J2	88°	103°	Open joint
J3	50°	198°	Open joint
J4	83°	190°	Open joint
J5	65°	133°	Open joint

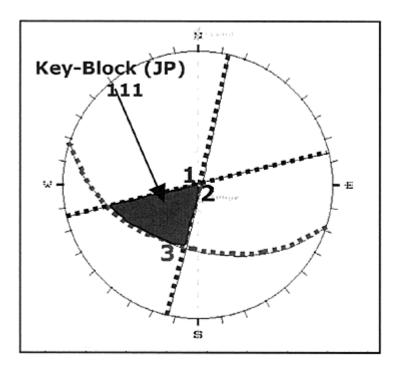


Figure 1. Stereographic projection of unstable block, intersection of Calle 7 (C-7) and Zanja 30 (Z-30), Reservas Norte.

Weight (ton)	Volume (m ³)	Apex (m)	FOS	ESF
191	70	6.8	0	100%
Failure Mode: Fall/Li	ft			
Area J1=44 m ²		Basal area =31 m ²	2	
Area J2=39 m ²		Note: FOS is facto	or of safety and	
Area J3=46 m ²		ESF is the excess	sliding force.	

Table 2. Output information of failed block at intersection of Calle 7 and Zanja 30, production level of Reservas Norte.

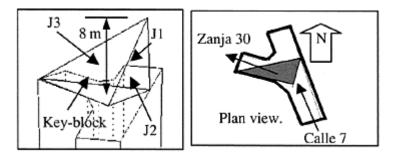


Figure 2. Estimation of block volume using Unwedge 2.2.

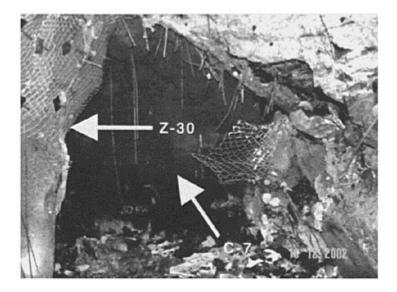


Figure 3. Overbreak generated at the roof because block failure at the intersection of Calle 7 (C-7) and Zanja 30 (Z-30), Reservas Norte.

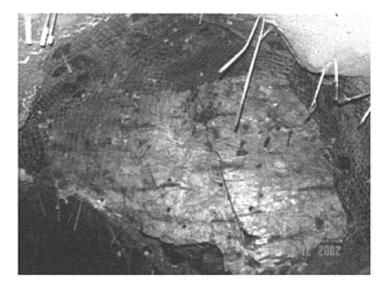


Figure 4. Part of the block that did not fall at the intersection of Calle 7 (C-7) and Zanja 30 (Z-30), Reservas Norte.

The following lines show an example of the type of analysis developed with key-block softwares.

Example responds to the intersection of two excavations (Calle 7 and Zanja 30), at the production level, Reservas Norte Sector. Section of galleries is 4 m by 4 m, with trend of 165° and 105° for Calle 7 (C-7) and Zanja 30 (Z-30), respectively.

4.3 Sensibility analysis of key-block stability

This analysis is carried out based upon the strength and stress field acting on joints. The stability is analyzed only for a single joint, considering the MohrCoulomb criterion for strength, and calculation of stresses using Kirsch equations, therefore, the stability of the entire block is not considered. The idea is to identify the stress field condition around the joint that determines it failure, and to extrapolate this condition to the failure of the entire wedge.

Different combinations of stresses and resistant properties of joints are used, considering a wide range of vertical and horizontal stresses around the excavation (assuming a biaxial field of stress), combined with a complete range of properties: low, medium and high strength, for medium size joints and major (principal) structures. In the analysis are fixed the resistance properties of joint fillings while evaluating stability at different stresses. Figure 5 shows a scheme of the sensibility analysis.

4.3.1 Assumptions

The assumptions at this stage of the analysis are the following:

- The strength of joints is calculated with MohrCoulomb criterion, using cohesion and friction properties of joint fillings.
- Estimation of normal and shear stresses acting on the joint, and shear strength relative to joint plane is made with Kirsch equations (elastic theory), considering a circular opening in a medium subject to biaxial field of stress. This is analog with respect to a problem of a long excavation at high depth, in a massive elastic rock, subject to isotropic conditions. Figure 6 shows diagram of stresses and the equations related to Kirsch (1898) are shown in Equations 1, 2, 3 below:

$$\sigma_{\Gamma} = \frac{\sigma_{V}}{2} \times \left\{ \left(1 + k \right) \times \left(1 - \frac{a^{2}}{d^{2}} \right) - \left(1 - k \right) \times \left(1 - \frac{4 \times a^{4}}{d^{4}} + \frac{3 \times a^{2}}{d^{2}} \right) \times \cos(2 \times \beta) \right\}$$
(1)

$$\sigma_{t} = \frac{\sigma_{V}}{2} \times \left\{ (1+k) \times \left(1 + \frac{a^{2}}{d^{2}} \right) + (1-k) \times \left(1 + \frac{3 \times a^{4}}{d^{4}} \right) \times \cos(2 \times \beta) \right\}$$
(2)

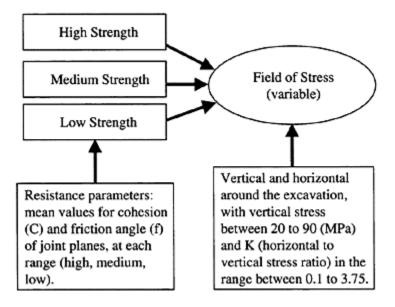


Figure 5. Sensibility process for joint stability.

$$\tau_{r\theta} = \frac{\sigma_{V}}{2} \times \left\{ (1-k) \times \left(1 - \frac{3 \times a^{4}}{d^{4}} + \frac{2 \times a^{2}}{d^{2}} \right) \times \sin(2 \times \beta) \right\}$$
(3)

4.3.2 Definition of input parameters

- Field of stress: in this analysis a bi-axial field of stress is considered, with a range of vertical stress between 20 to 90 (MPa) and horizontal to vertical stress ratio K (σh/σv), between 0.1 to 3.75.
- Resistant properties (
 ^{(Φ} and C) for joint fillings: Table 3 shows strength of fillings for medium size and principal joints analyzed in this paper.
- Geometric parameters: this type of parameter is defined through the analysis of failed key-block. A summary of these properties is presented in Table 4.

In all the studied cases, wedge size is coincident with the maximum block formed at excavation's roof

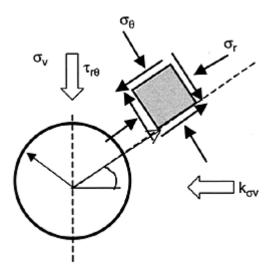


Figure 6. Diagram of stresses around a circular excavation.

Table 3. Resistant properties (ϕ and C) for joint fillings.

Strength (major) size	Туре	Med size	ium	Princ joints	ipal (maj	or)
		φ	С	ф	С	
High	Quartz	30	8		30	3
Medium	Chalcopyrite, an-hydrite, feldespate, calcite, etc.	22	3		18	1
Low	Clay, chlorite, talc, gypsum, etc.	10	0.1		8	0

Note: Medium size joints are the persistent geological structures with a range of length of 5 m to 15

m.

Principal size joints are the persistent geological structures with a range of length of 50 m to 150 m. Note: All the analysis considers the cohesion and friction angle of andesite, which are 5 (MPa) and 32°, respectively.

or wall. It is important to note that ranges of geometric parameters in Table 4 are only general and preliminary results.

- From Table 4 is possible to depict a range of d/ae between 2 and 3, giving an average value for d/ae=2.5. This range is coincident with the maximum ratio of shear and normal stresses acting on the joint. An example of this is shown in Figure 7, for k (horizontal/vertical stresses) equal to 0.5. Therefore, in the analysis the stresses acting on the joint are obtained fixing a value of d/ae=2.5, which is substituted on Kirsch equations.

Case Se S V Α В EA D d/a_e ae 6×6 6 45 3.6 30 14.3 2.15.7 2.70 6 46 4.4 31.4 32 3.2 6×6 7.6 2.38 10×4 10 70 6.8 31 38.3 3.5 10.3 2.94

30

46.3

3.7

7.8 2.13

Table 4. Summary of geometric parameters of failed blocks.

Where:

1

2

3

4

Se: section of excavation, is the width or length and the height of the tunnel $(m \times m)$.

4.1

S: maximum span of the excavation, which responds to width or length (m).

V: volume of the block (m^3) .

A: maximum apex of the block (m).

12×4

B: basal area of the block (m²). Is the area defined in the intersection of the excavation wall or roof, and the block.

EA: excavation area (m^2) . Is the area of the cross section opening.

12 41

ae: equivalent radius for the excavation cross section (m).

d: distance from the center of excavation where the stresses are calculated=ae+A.

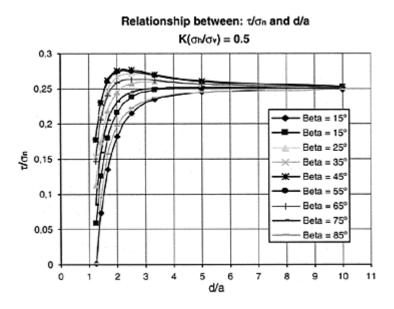


Figure 7. Relationship between ratio of shear and normal stresses acting on the joint and ratio of distances from the center of excavation. Beta is dip angle of the joint.

4.3.3 Results

Figures 8 to 13 shows stability graphs of joints subjected to biaxial field of stress around the excavation. At each graph the x-axis is the field stress ratio (horizontal stress/vertical stress), while the y-axis is the factor of safety for each value of x. Values of x are plotted as a function of vertical stress encountered at the mine, so different curves are depicted. The graphs are zoned in 4 categories according to the states of the joint: unstable zone for FS< 1.0, fair zone for 1.0>FS<1.5, stable zone for 1.5>FS< 2.5 and very stable zone for FS>2.5.

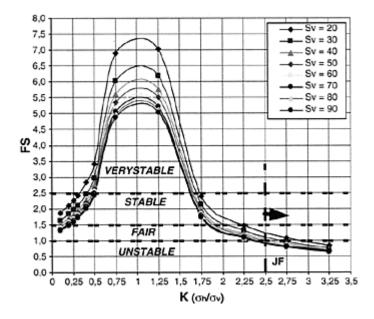
Figure 8. Stability Graph for Medium Size Joints with High Strength Filling: at the left side of the graph there is a tendency to joint failure (JF) at low values of k, for high vertical stresses (80–90 MPa). Is not erroneous to say that failure occurs for k<0.05, for vertical stresses between 80–90 MPa. At the right side of the graph, JF begins at K=2.5 for σv >50 (MPa).

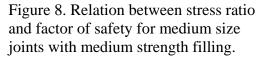
Figure 9. Stability Graph for Medium Size Joints with Medium Strength Filling: at the left side of the graph, JF begins at K=0.1 for σv >50 (MPa). At the right side of the graph, JF begins at K=2 for σv >40 (MPa).

Figure 10. Stability Graph for Medium Size Joints with low Strength Filling: at the left side of the graph, JF begins at K=0.5. At the right side of the graph, JF begins at K=1.65.

Figure 11. Stability Graph for Major Size (Principal) Joints with High Strength Filling: at the left side of the graph there is a tendency to joint failure (JF) at low values of k, for high vertical stresses (80–90 MPa). Is not erroneous to say that failure occurs for k<0.05, for vertical stresses between 80–90 MPa. At the right side of the graph, JF begins at K=2.25 for σv >50 (MPa).

Figure 12. Stability Graph for Major Size (Principal) Joints with Medium Strength Filling: at the left side





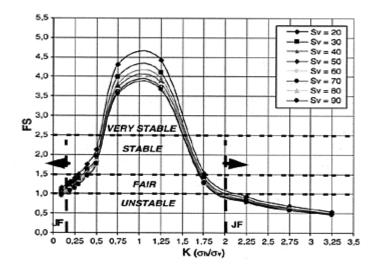


Figure 9. Relation between stress ratio and factor of safety for medium size joints with medium strength filling.

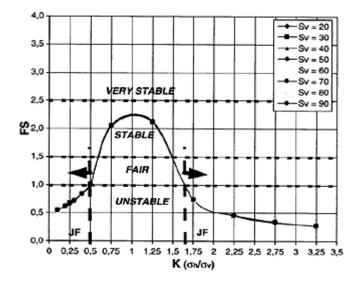


Figure 10. Relation between stress ratio and factor of safety for medium size joints with low strength filling.

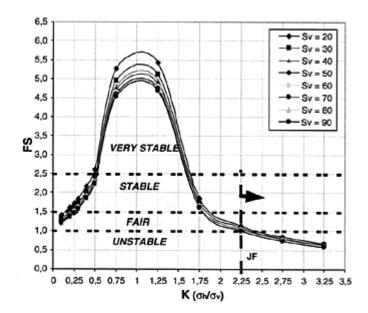


Figure 11. Relation between stress ratio and factor of safety for major size (principal) joints with high strength filling.

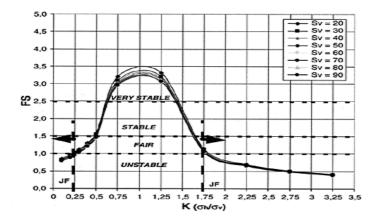


Figure 12. Relation between stress ratio and factor of safety for major size (principal) joints with medium strength filling.

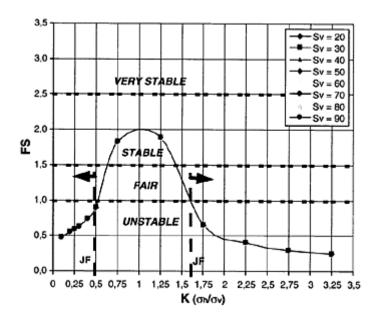


Figure 13. Relation between stress ratio and factor of safety for major size (principal) joints with low strength filling.

of the graph, JF begins at K=0.25 for σv >50 (MPa). At the right side of the graph, JF begins at K=1.75.

Figure 13, Stability Graph for Major Size (Principal) Joints with Low Strength Filling: at the left side of the graph, JF begins at K=0.5. At the right side of the graph, JF begins at K=1.6.

In general terms is important to note that the curves for vertical stress have less convergence in the range k=0.5–1.5, coincident with the most stable zone in the graphs. In the other hand, there is more convergence in the sides of the graphs, for k<0.5 and k>1.5. In addition, when strength properties of joints decrease, an increase of convergence for the vertical stress curves occurs. Even, for joints with low strength of fillings, factor of safety (FS) only depends on stress factor (k), independent of vertical stress magnitude (σ v).

It is possible to establish that changes in the condition of stresses controls stability around the excavations. This is an important issue for predicting joint or block stability according to different state of stresses through the mining process.

4.4 Case study

Case study corresponds to block fall in XC-30 Fw, ventilation level of Reservas Norte Sector, which is analyzed with more detail to find out condition of stresses (stress ratio and stress magnitude) that determine wedge failure.

4.4.1 Reservas Norte Mine

Reservas Norte Sector is one of the newest mining projects developing at El Teniente. At the moment is formed by two mining areas (sub-sectors), an old one corresponding to Sector Sub-6 Area Invariante (in production since 1991, and a new one, Area Andesita Sector, beginning production during year 2004). A third sub-sector called Pilar Sub-6/Esmeralda, is still subjected to engineering studies, hoping to begin preparation during year 2004. Production rate during year 2004 is in the order of 20,000 ton/day hoping to produce 40,000 ton/day on year 2009. Caving method corresponds to pre-undercut panel caving method.

4.4.2 Location and general information, case study

This case corresponds to a block fall located at XC-30 Fw (in "Sector Hw, or West Sector"), ventilation level (2083 above the sea), 20 meters under production level, of Reservas Norte Area Invariante. In this sector, coordinates N: 658, E: 800, was developed a reparation process at the segment of excavation damaged by rockburst occurred on April-22–2003. This process included re-supporting with: fully grouted rebar (Φ 22 mm), pattern 1 m×1 m, 3 meters of length, chain-link mesh (10006), shotcrete 10 cm and also installation of fully grouted long cables bolts (Φ 15.2 mm), 10 meters of length, at the position of G and F Faults. Is important to note that old support (installed at year 1988) consists of fully grouted rebars, pattern 1 m×1 m and 2.1 m of length, welded mesh and shotcrete lining (10 cm of thickness) was installed at damaged zone.

At the position of G Fault, during the reparation process, overbreak associated to this fault system occurs, associated to a block failure with 3 to 4 meters of apex.

4.4.3 Geology

Lithology corresponds to Primary Andesite, with the presence of important Faults Systems such as G and F Faults Systems, in the zone.

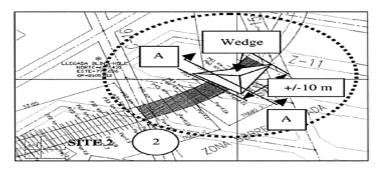


Figure 14. Location of block fall at XC-30 Fw, ventilation level, Reservas Norte.

4.4.4 Stress field

The affected zone is located in the environs of the caving front, in the abutment zone of stresses. According to information of stresses from hollow-inclusion monitoring cells, installed at Reservas Norte "Sector Hw", σ_1 is in the order of 60–70 (MPa), σ_2 in the range of 40–50 (MPa) and σ_3 between 15–20 (MPa).

4.4.5 Damage before block failure

Sector was affected by four rock-burst between years 1990 and 2003. The level of damage was classified in the range low to medium, corresponding to minor spalling and bulking of rock and shotcrete lining, up to block failure of 1 m of apex, respectively. Last rock-burst damage (April 22, 2003) was classified as medium, covering up to 50% of tunnel section (6 m×6 m, horse-shoe geometry), with 1 m³ size of blocks. Key-block falls during the reparations of the zone (on September 2003), after last rock-burst.

4.4.6 Damage generated by block (wedge) failure

Damage induced by block fall, occurred during the reparation process in the sector, involves 4.5 meters of overbreak and sloughage at excavation's roof, roughly 10 metres along the tunnel. Location of block fall and section describing overbreak and sloughage at this sector are shown in Figures 14 and 15.

4.4.7 Block geometry

Block geometry is generated through key-block analysis developed with PT-Tunnel Software, as described in Figure 16 and Table 5. According to this analysis key-block has a volume of 46 m^3 (124 ton of weight), and a maximum apex of 4.4 meters.

4.4.8 Back-analysis of block failure

At previous stages of this paper, hypothesis of block stability depending principally upon changes in field stress was introduced. Mechanism of block failure

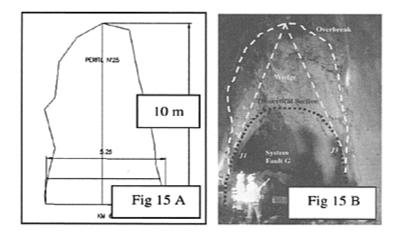


Figure 15 (A and B). Cross section A-A (from Figure 14) of XC-30 Fw in the zone of wedge failure.

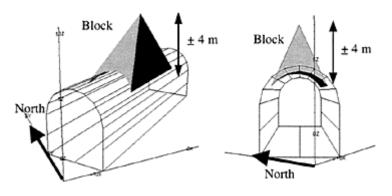


Figure 16. Estimation of block volume using PT-Tunnel Software.

Table 5. Output information of failed block at intersection of XC-30 Fw, ventilation level of Reservas Norte.

Weight (ton)	Volume (m ³)	Apex (m)	FOS	ESF
124	46	4.4	0.01	99%
Failure Mode: Fall/	Lift			
Area J1=29 m ²		Area J5–19 m ²		
Area J2=4 m ²		Basal area=31 m ²		

Area J3=44 m ²	Note: FOS is factor of safety and
Area J4=0 m ²	ESF is the excess sliding force.

is analyzed for the case study, considering possible changes on field stress ratio.

Figures 17 and 18 show historic stress ratio obtained from monitoring data at Site 2 (depicted with a circle at Figure 14). This site is located roughly 40 meters to the south and 20 meters above the damaged zone of XC-30 Fw, in Calle 16, between Zanjas 9 and 10, production level of Reservas Norte Area Andesita.

At Figures 17 and 18, in x-axis is depicted the day of monitoring, beginning from day 1 (on January 1999), and finishing day 39 (on October 2003), so

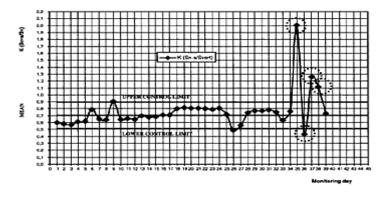


Figure 17. Graph of registered horizontal stress (northsouth) and vertical stress ratio, at Site 2 Reservas Norte.

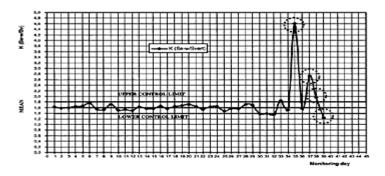


Figure 18. Graph of registered horizontal stress (northsouth) and vertical stress ratio, at Site 2 Reservas Norte.

monitoring frequency varies between 1 to 2 registers per month. Vertical axis (y) is the ratio between horizontal and vertical stresses. Three horizontal lines are depicted at each graph, indicating: mean value of K (central line), mean plus 2 times standard deviation (upper control limit), and mean less 2 times standard deviation (lower control limit). Mean value for K1 (north-south/vertical stresses) is 0.71 with a standard deviation of 0.09. Mean value for K2 (east-west/vertical stresses) is 1.59 with a standard deviation of 0.11.

From these graphs is possible to note that records outside the range of control limits are encountered. These values are pointed out with circles in Figure 17 and 18, and are shown in Table 6.

To find out the stress ratio that determines stability at each joint plane, a decomposition of horizontal stress vectors have to be done in the direction of dip direction of each plane (J1, J3, J5) that conforms the wedge, as illustrate in Figure 19. Joint characteristics are shown in Table 7.

A summary about the vertical stress and stress ratio (horizontal/vertical) acting on wedge joints is shown in Table 8, for the records outside the control limits defined in the graphs.

Monit. day	Date	K1 (north-south)	K2 (east-west)
35	May 05/03	2.0	4.6
36	June 12/03	0.4	1.6
37	July 30/03	1.3	2.7
38	Sept 11/03	1.1	2.0
39	Oct 31/03	0.7	1.2

Table 6. Records out of control limit.

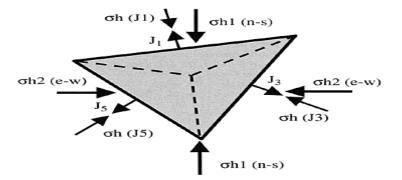


Figure 19. Decomposition of horizontal stress vectors in the direction of dip direction of each plane of the wedge.

Joint	Id.	Dip (°)	Dip Dir	(°)	Туре	:			
1	J_1	85	340		Faults	s with medium t	o low streng	th of filling	g
3	J_3	70	145						
5	J_5	45	235						
		,			,	acting on w	0.0		
			aphs.	usiue	e the C	control limit	s defined	in the	
Monit.	day		aphs.	K1	K2	control limit σv (MPa)	k (J1)	k (J3)	k (J5)
Monit. 35	day	gr Date	aphs.						k (J5) 4.92
	day	gr Date May	aphs.	K1	K2	σv (MPa)	k (J1)	k (J3)	
35	day	gr Date May	aphs. 05/03 12/03	K1 2.0	K2 4.6	σv (MPa) 16.5	k (J1) 4.28	k (J3) 4.67	4.92
35 36	day	gr Date May June	aphs. 05/03 12/03 30/03	K1 2.0 0.4	K2 4.6 1.6	σv (MPa) 16.5 41.9	k (J1) 4.28 1.25	k (J3) 4.67 1.41	4.92 1.52

Table 7. Characteristics of Joints J₁, J₃, J₅.

4.4.9 Discussion of results

- Joints that form the block in this case study are faults with medium to low strength of fillings. For this type of structure, according to joint stability graphs, Figures 8 to 13, failure occurs when K is greater or equal to 1.8 and when k is less or equal to 0.5. Calculated values of k outside control limits are compared with the threshold defined for the structures. Cells filled with gray color at Table 8 indicate values of k where failure occurs. The date of this failure is posterior to the occurrence of rock-burst on April-22–2003, and can be associated to a seismic stress ratio that control wedge fall. In other words wedge fall is coincident with a high horizontal to vertical stress ratio, greater than 2 (cells depicted in yellow).
- Is possible to conclude that installed support in the zone of wedge, consisted of fully grouted rebars, pattern 1 m×1 m and 2.1 m of length, welded mesh and shotcrete lining (10 cm of thickness), was insufficient to hold up the block. No cables where installed at that time.

5 CONCLUSIONS

- In all the studied cases, wedge size is coincident with the maximum block formed at excavation's roof or wall. As general and preliminary results, is possible to establish that apex/span ratio moves in the range 0.34 to 0.7 while volume/apex ratio is in the range 10 to 13 (m²). This type of information could be useful in the design of future

reinforcements for excavations, when few or no geological information exists. Nevertheless is necessary recovering more information of fallen blocks and developing more accurate studies, with the objective of delimit the geometric characteristics of key-blocks.

- A mechanism of block failure controlled principally by an unfavorable induced stress ratio is introduced in this paper. Stability of block is analyzed through the analysis of each joint, so stability graphs for joints with different strength of fillings are encountered.
- Stability graphs for joints show that failure zones are delimited at both sides of the graphs. At the left, joint stability is defined by K value (horizontal stress/vertical stress) less than 0.5, determining stress relaxation or reduction of horizontal (clamping) stress with respect to vertical stress. In the other hand, zone depicted at the right side of the graphs, indicates that joint stability is controlled by a value of k greatest than 1.7, in other words horizontal stress greater than 1.7 times vertical stress.
- In general terms is important to note that the curves for vertical stress have less convergence in the range k=0.5–1.5, coincident with the most stable zone in the graphs. In the other hand, there is more convergence in the sides of the graphs, for k<0.5 and k>1.5. In addition, when strength properties of joints decrease, an increase of convergence for the vertical stress curves occurs. Even, for joints with low strength of fillings, factor of safety (FS) only depends on stress factor (k), independent of vertical stress magnitude (σv).
- Applying the threshold values obtained from joint stability graphs into the case study, it is possible to note that wedge fall is coincident with a high horizontal to vertical stress ratio, greater than 2. In addition, support installed in the zone of the wedge was insufficient to hold up the block.
- It is important to note that joint stability graphs are the first step of a more extensive study, so is recommendable developing numerical models using stress ratio presented in the graphs, with the objective of calibrate them and building up stability graphs that represent the condition of the entire wedge.

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Quality in ground support management

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ABSTRACT: A quality assurance program in ground control management contributes to improvement in safety and productivity. The program is achieved by developing and implementing a company ground control policies and management system. The system is effective when authority and responsibilities are clearly specified and delegated to a competent person(s). The system should be documented in a Ground Control Management Plan. The Plan must specify agreed actions and a method of approval and verification of ground control activities.

Where ground stability is affected by deterioration of support or deterioration in ground conditions, or support does not comply with current standards, existing excavations should be additionally supported, re-supported or rehabilitated. To ensure safety of all underground personnel a minimum support standard must be adopted. The approved minimum support requirements vary depending on size and shape of an excavation, expected service life (stand up time), usage (often visited or travelled through, storage of material, etc).

All ground control activities such as data collection, drilling, blasting, maintenance of excavations and monitoring should be described in the work procedures. Quality assurance requires that a regular inspection and monitoring program should be established, carried out by competent persons and conducted for all areas identified during risk assessment.

1 INTRODUCTION

Ground instability in underground mining excavations often results from deficiency in quality in ground support activities. It often happens that these activities have been developed by the past experience to meet local ground support requirements. The activities may not follow recent research developments and may be deficient in certain areas, e.g. geotechnical management systems, document control, communication, geotechnical input into mine design, support installation or in geotechnical risk management (Szwedzicki, 1989).

Safety and cost efficiency of mining operations could be substantially increased by implementation of a policy on Quality Assurance in Ground Support Management. Increased safety in mines and improved rock mass stability result from improved geotechnical design process, efficient monitoring, inspection and reporting, and increase in quality of support.

The main objective of a quality system in ground support management is to produce information and then use that information for improvement in mine safety (through increase in stability of the rock mass), and reduction of production (mining and milling) costs (Szwedzicki, 2003).

The International Standard Organisation 9000 Series (ISO 9001, ISO 9002 and ISO 9003) provide guidance in developing an effective quality system that can be integrated into a geotechnical management system.

2 QUALITY IN GROUND SUPPORT

Ground support (including rock support and reinforcement) activities consist of recognition of ground conditions, support design, support installation and monitoring of support elements and rock mass performance. Quality represents features and characteristics of the activities that bear upon its ability to satisfy stated or implied needs. In simple terms, in relation to ground support, quality is conformance to requirements and specifications.

Quality control are the operational techniques and activities to fulfil requirements for quality while quality assurance is defined as planned and systematic actions to provide adequate confidence that activities will satisfy requirements for quality.

Stages of implementation of the quality program in ground support include (after ISO 9002):

- Preparation of a Policy on Quality Assurance in Ground Support

The quality policy refers to management commitment and is a description of company objectives. The policy objectives should be quantifiable and measurable and should form a part of the Ground Support Management Plan. The policy should be supported by procedures that are documented methods of carrying out ground support tasks. Policy and procedures should be relevant to the whole organisation and should be approved by an authorised person.

- Appointment of a management representative

A competent person(s) should be appointed and should be given authority and responsibility for all ground control activities.

- Development of a Ground Support Management Plan

The Ground Support Management Plan introduces quality measures in each step of the mining activities and should focus on key control points (i.e. data collection, design, installation, and ground support performance monitoring and geotechnical risks management). The Plan should specify responsibilities and geotechnical actions with time frames.

– Determination of objectives and targets

Work procedures are standard documents showing each step of a job or a task, the hazards identified with each step and actions to be adhered to in order to manage and control each hazard. Specific quality parameters and acceptance criteria should be included in all work procedures. The criteria should specify acceptable deviation from the standards.

- Implementation and control of all activities

For the system to be operational, all ground support activities must be implemented and controlled by periodical reviews or audits. Actions arising should be verified in the agreed time frame.

– Review

The Quality program should allow for feedback and be periodically reviewed. The reviews should be carried out using checklists of set targets and record the achievements and deviations from the targets. The program should promote and facilitate continuous improvement in ground support activities. The ground support quality system at a mine should comprise of geotechnical management (organisational structure, authorities and responsibilities), documentation (procedures, practices, instructions, and specifications), support installation and monitoring.

3 QUALITY ASSESSMENT

Quality can be described by a quality grade and a quality level (Fox, 1995). The grade is specified by standards or specifications, e.g. rock bolt protrusion of 0.1-0.3 m (low grade) or 0.10-0.15 m (high grade). The level can be specified by a number of bolts installed according to the specification e.g. 99 out of 100 (high level) or 90 out of 100 (low level).

Quality grade is defined in quality policy, procedures or specifications while quality level is achieved in an implementation phase, i.e. during geotechnical activities.

Measuring ground control quality can be achieved by recognising a shortfall in:

- 1. quality grade, i.e. discrepancy between company specifications (requirements or practices) and standards, best work practices or manufacturers specifications.
- 2. quality level i.e. discrepancy between the requirements set in geotechnical procedures or specification and actual implantation of the activities.

Such discrepancies can be recorded and counted as:

- number of deficiencies recognised (quality grade and level),
- number of items found deficient (quality level),
- measured deviation from the standard (quality level).

Assessments of ground support quality assurance can be achieved through audits and management reviews. An audit is a systematic and independent examination to determine whether activities and related results comply with planned arrangements and whether these arrangements are suitable to achieve objectives. The objective of an audit on quality assurance in ground support is to provide mine management with the information on status and potential improvement in ground support activities. The audit also covers safety and risk management aspects of mining operations related to the ground support management.

There are two types of audits—a system audit and a compliance audit. The system audit is used to determine the existence and validity of the ground support management system. The compliance audit is used to confirm whether or not specified procedural practices in geotechnical planning and design, ground support installation and inspection and monitoring are actually being implemented and are effective.

A system audit on the quality assurance in ground control management system should seek evidence of:

- clearly defined responsibilities and authorities,
- documented procedures, practices and instructions,
- knowledge and understanding of responsibilities, authorities, procedures, instructions, etc.

A compliance audit on quality assurance in ground support management system should seek evidence of:

- correct operational procedures approved by the authorised person,

-adequacy of personnel, equipment facilities and general resources,

-effectiveness of the system when correctly operated.

The audit cannot examine all activities but chooses random samples and examines them for non-compliance or for possible improvement. The audit is not an appraisal activity or process but an action taken to prevent the recurrence of any deficiencies discovered. When the evidence collected indicates that the requirements of the procedures and standards are not being followed, this should be recorded as non-conformance. Management reviews and audits can highlight areas for potential improvement. Action plans are then developed to address identified issues.

A program of the audit on quality assurance in ground support management could include the following techniques:

- interviews and discussions with line managers responsible for ground support activities,
- inspections of support installation places underground mines,
- review of ground support documents, standards, work procedures and practices,
- discussion on and review of geotechnical input into support design,
- observation and monitoring of quality of drilling and blasting,
- observation of rock mass behaviour and modes of failure,
- estimation of ground support long term performance,
- interviews and discussions with supervisors and operators responsible for scaling and support installation,
- review of geotechnical records and data,
- discussions with geotechnical, mining production and other technical staff,
- development of action plans to address identified issues.

The audits can highlight the present achievements that meet international mining standards but also may disclose deficiencies and may reveal directions for further improvement.

4 QUALITY ASSURANCE IN GROUND SUPPORT MANAGEMENT SYSTEM

The Ground Support Management System should ensure that an organisational structure exists to manage and verify quality in ground support. A management system can be evidenced by the presence system components described in the following sections.

4.1 Responsibilities and authority

A competent and suitably qualified person(s) must be appointed to manage, supervise and perform ground support activities such as assessment of ground stability, support installation, maintenance of excavations, and support performance monitoring. Responsibilities and authority must be well defined and should be reflected in respective job descriptions.

4.2 Compliance with mining legislation

A management system must be created so that all ground support activities are carried out in compliance with mining safety legislation. The system has to ensure that provisions of the Act and Regulations are followed.

4.3 Competency and training

A competency and training system must be developed to ensure that all employees responsible for ground support activities are competent (i.e. trained and qualified) in reading ground conditions, detecting signs of ground instability and carrying out scaling of the exposed ground. It must also ensure that supervisors and professionals are continuously trained and exposed to newly introduced procedures and practices. Relevant records of education and training should exist for all employees involved in ground support.

4.4 Communication and reporting

A communication and reporting system between various levels of a management structure and between all professionals involved in ground support issues should be established and enforced. Communication can be formal (e.g. written instructions, memoranda) or informal (e.g. verbal during meetings). The communication system should ensure that all interested parties receive the required information and that information is understood. Proper communication channels should allow for effective feedback.

4.5 Document control

A document control system requires that all ground support policies and procedures are approved, distributed, reviewed and archived. Document distribution and circulation must follow an approved list to ensure that all relevant personnel are advised and have been provided with access to relevant documents. The system must prevent documents from being withheld or put aside.

A suitable person should be responsible for revisions and must implement a system that allows for timely withdrawal of old documents and replaces them with the latest versions. All written procedures and practices should have a revision date by when they must be discussed and reviewed. If needed they should be updated or modified. All changes must be readily available and be communicated to personnel that might be affected.

A record keeping and archiving system has to be developed to prevent misplacement or loss of documents, consultants' reports, collected data, etc. Records should be readily accessible when required.

5 QUALITY ASSURANCE IN GEOTECHNICAL PLANNING AND DESIGN

Geotechnical design criteria should be established and a system of geotechnical input into support design should be implemented in the early life of the mine. Results of the investigations into ground conditions should form a base for support selection and design.

5.1 Data collection and analysis

A system should be in place to ensure that all needed geotechnical information and data are systematically collected, processed, interpreted, analysed, documented and archived. The information and data should be collected according to accepted standards or well established methods. Changes in ground conditions or behaviour have to be monitored and reported. The collected data and information must serve a purpose and must be analysed. The results should be used for design, planning, and installation of ground support.

5.2 Geotechnical planning

A systematic approach to mine planning and design should be based on geotechnical engineering methods. Geotechnical planning should take into account the life of each excavation and life of the mine.

5.3 Geotechnical design

A responsible person must ensure that ground support design is based on appropriate geotechnical information and takes into account geotechnical risks. A system must ensure that geotechnical design parameters are used to optimise the size, shape and orientation of mining excavations. Geotechnical design parameters established for each rock mass

domain should form a base for assessment of rockmass stability. Geotechnical considerations should be given to determining maximum open span of excavations. The effect of interaction of excavations and backfill should be considered. Potential for mining induced seismic activity should be considered.

5.4 Approval system

Ground support information should be prepared and documented by a competent person and must be approved by an authorised person. Variation from the original design should be documented and approved.

5.5 Feedback and follow up

A ground support management system should establish procedures for verification of plans and design as a project progresses and rock mass behaviour changes. Geotechnical ground support performance monitoring should provide further information and feedback for successful implementation of findings into an evolving plan of remedial measures. The feedback might be used to modify the design or change some design parameters in the consecutive design phases.

6 GROUND SUPPORT MANAGEMENT PLAN

A Ground Support Management Plan should be prepared and approved. It should also be reviewed and updated periodically. It is a leading geotechnical document that describes ground support system and includes all relevant geotechnical information. The document should include important geotechnical data, specify minimum standards of ground support, refer to procedures and consultants' reports, and give an overview of geotechnical settings, e.g. classification rock mass and delineation of geotechnical risk. The document should include a schedule of ground support performance monitoring, and short and long term plans of geotechnical activities. The Plan has to be approved by management and reviewed annually or more frequently if necessary.

A Ground Support Management Plan should cover the whole-of-mine life, including a mine closure phase. Provision for geotechnical aspects of mine closure should be addressed, e.g. long term stability, securing the access to the excavation, and possible future use of excavations.

Geotechnical requirements for minimum ground support in terms of support pattern, mesh overlap, properties of grout and thickness of shotcrete are specified in installation standards. The installation standards can be adopted according to geotechnical conditions.

6.1 Minimum ground support requirements for underground excavations

The minimum support requirement must form a part of the Ground Support Management Plan. Rock mass support recommendations must conform with requirements or specifications provided in legislation, company documents such as procedures, rules, standards, codes of practice or approved best work practices or suppliers/manufactures' guidelines (Szwedzicki, 2004). Figure 1 gives an example of not adhering to ground support instructions. In highly corrosive environment, black and galvanised plates were used together with galvanized friction stabilisers.

In adverse geotechnical conditions e.g. weak ground, presence of structural features, expected corrosion, or mining induced stress, the minimum support standard should be reviewed and, if needed, additional support should be recommended and installed.

To ensure safety of all underground personnel a minimum support standard must be approved. Minimum support requirements must be provided for all excavations such as: ramps, accesses, drifts, drives, galleries, crosscuts, chambers, workshops, etc. The approved minimum support requirements may vary depending on size and shape of an excavation, expected service life (stand up time), and usage (often visited or travelled through, storage of material, etc.).

Support recommendations have been developed by experience gained in excavations of underground mines and they take into account:

- geotechnical conditions,
- exposure level (frequency of use and purpose),
- size of the excavation,
- service life of the opening,
- mining induced stress,
- potential for corrosion.

For all classes of minimum ground support, details of pattern and installation procedures should be specified in Support Installation Standards.

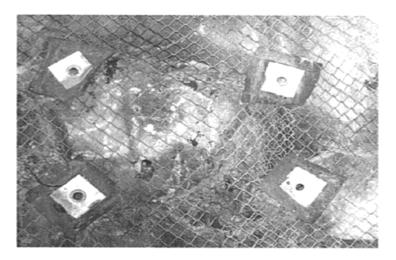


Figure 1. Black butterfly plates used with galvanised dome plates in highly corrosive environment.

6.2 Existing mining excavations

All accessible excavations shall be periodically inspected to review ground support status and stability of all underground horizontal excavations. If support doesn't meet the present required standard, geotechnical recommendations shall be issued on additional ground support and the priority of repair/ rehabilitation works shall be specified.

Existing mine excavations, some of them developed many years ago, might not have been supported at all or were supported to now superseded requirements that were in force at the time. It is possible that what was acceptable in the past, during a development phase, does not meet current safety standards nor geotechnical requirements. Ground instability in existing mining excavations often results from deficiency in ground support management i.e. geotechnical data collection, design of support, installation and monitoring. For example, ground support procedures might become inadequate to manage risk over time, improper practices were accepted or faulty materials and equipment was used. Additionally both geotechnical and mining factors could have changed during the life of underground excavations. Extended stand up time of excavations often results in changes to geotechnical and mining conditions such as:

- reduction in mechanical properties of the rock mass due to weathering, changes in mining induced stress and/or water inflow,
- deterioration of supporting ability of reinforcing elements due to corrosion, blast and mechanical damage, Figure 2,
- changes to the geometry of existing openings due to alterations in mine design or deterioration of ground.

As a result of changes in geotechnical conditions, the level of risk in existing mining excavations is changing and may result in uncontrolled ground

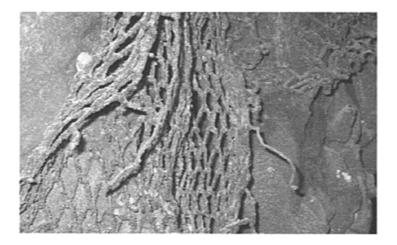


Figure 2. Corrosion of mesh in an existing excavation.

movement in the form of rock falls. For all existing excavations that may extend hundreds of kilometres in large mines, support requirements should be systematically assessed for risk of potential fall of ground. Each area should be periodically inspected for stability and support requirements with special attention being paid to areas where:

- ground conditions are likely to deteriorate over time,
- loose material is being generated and constantly requires scaling,
- mining induced stress may increase or decrease, or there are significant blast vibrations,
- corrosion of support elements can take place.

7 REVIEW OF SUPPORT REQUIREMENTS

The review of geotechnical support requirements aims at gathering geotechnical information and using that information to improve mine safety through increasing stability of the rock mass. Areas for support review can be identified in two ways-by systematic inspections by geotechnical personnel or by reports from other operators at the mine. The objective of support review is to provide mine management with information on the status and potential needed for additional support, repair or rehabilitation of ground support. Where ground stability was affected by deterioration of support, or by deterioration in ground conditions or where support does not comply with the current standards, excavations should be additionally supported (e.g. mesh pinned to the existing bolts), re-supported (e.g. replacing corroded bolts) or rehabilitated (old support removed and new support installed). Some excavations may need periodical scaling and parts that might be unsafe for entry should be barricaded off. The risk assessment should be documented in a format that will enable an action plan to be developed. As a result of risk assessment, rock mass stability should be reassessed and recommendations made. The geotechnical risk assessment should set out strategies to ensure that minimum ground support requirements are met. These strategies include:

- reviewing established rules on ground support,
- defining minimum ground support requirements and standards for all existing excavations,
- specifying which openings are to be maintained for present and future mining activities,
- carrying out underground inspections and review of the present state of support.

The following steps should be followed to ensure the success of the support requirement review:

- nomination of reviewing teams and establishing review criteria,
- assigning areas of responsibility for carrying out inspections,
- inspection and monitoring,
- carrying out risk analysis and recommending support work,
- installation of additional support to rehabilitate existing excavations.

In the case of existing excavations, areas of the mine that are identified (from the geotechnical risk assessment) as requiring rock support should be prioritised for remedial work and scheduled accordingly to the risk ranking. Prioritisation criteria take into

account safety, time and stress related ground deterioration and production requirements. Priority can be classified as:

- Priority 1 (critical)—support to be installed or rehabilitation carried out as soon as possible. This is recommended for areas that are unsafe and/or holding up production or development
- Priority 2 (high)—support or rehabilitation work to be included in the next month plans and to be completed within two months. The ground conditions in the recommended area are expected to deteriorate
- Priority 3 (standard)—support or rehabilitation work to be carried out as soon as reasonably possible and to be completed within three months. The area may deteriorate due to changes in ground conditions.

Ground conditions may deteriorate due to change in mining induced stress and are time dependent. Once deterioration starts it usually accelerates in time, leading to rock falls or closure of openings. It is crucial that support recommendations are implemented as quickly as possible.

8 QUALITY ASSURANCE IN GROUND SUPPORT ACTIVITIES

Ground support activities include preparation for support installation (support hole drilling and barring down), installation of support elements and support performance monitoring maintenance. Regular and systematic calibration checks should be made on the equipment used for drilling, support installation and geotechnical instrumentation.

8.1 Preparation for support installation

Before support elements are installed, preparation for installation has to be conducted. In new development they include scaling (barring down) and drilling hole for bolt installation. In existing excavation when ground support rehabilitation is carried out, preparation additionally may include: removal of old support elements, cutting old mesh to remove hanging rocks, additional blasting to remove lage potentially unstable blocks, etc.

8.2 Barring down

A written procedure on barring down (scaling) should specify the method of scaling rocks down and proper use of the equipment. For the existing excavations, the procedure should specify the minimum intervals between barring down as the rock mass can suffer damage from blasting and ground conditions may deteriorate with change in mining induced stress or time.

8.3 Drilling

Drilling procedure and specification for ground support installation, should be prepared and approved. The specification should include: collaring position, direction of drilling, hole diameter and hole length. Figure 3 gives an example of a 72 mm (instead of required 42 mm) diameter hole drilled for a 20 mm thread bar. The direction of drilling and hole length should be provided with acceptable deviations. The depth of the holes should be monitored and corrected as required. Drilling conditions should be monitored and a formal feedback should be provided to support engineers. That feedback, if required, should be used to modify design parameters.

8.4 Ground support

Quality assurance in ground support (including rock support and reinforcement) should be executed in design, installation and performance monitoring.

8.4.1 Ground support design

The design document should specify: type of support and reinforcement (e.g. length, diameter, steel type,



Figure 3. A 72 mm hole drilled for a 20 mm threadbar.

type of grout), support density and support layout (e.g. number of bolts in a row, spacing between the rows), and support specification (e.g. bolt hole position, inclination and

depth, thickness of shotcrete, mechanical properties of support material, consistency of grout). Support design should take into account mechanical properties of the rock mass, structural features of the rock mass, in situ and mining induced stress, and the effect of water on stability of the rock mass and on corrosion of support elements. Areas that are recommended for support should be indicated on mining plans. Figure 4 illustrates support installed without a formal geotechnical design.

8.4.2 Ground support installation

All support must be installed according to the design pattern and installation procedures. Procedures shall include details on: storage and handling of ground support material, assessment of ground support stability, ground support installation, and recording of installation data (Mines and Aggregate Safety and Health Association, 1998). Figures 5, 6 and 7 give examples of lack of quality assurance in installation of ground support.

8.4.3 Performance of ground support

A quality control program to assess the performance of installed support should specify the parameters and the conditions of testing. Performance of ground support should be tested after installation and then monitored over the life of mining excavations. Support that is to be tested in a destructive way should be installed in addition to the support required for the specific pattern.

Testing during or after the installation should include:

 Testing support elements to ensure that they meet specifications, e.g. consistency and properties



Figure 4. Over supporting of an underground excavation.

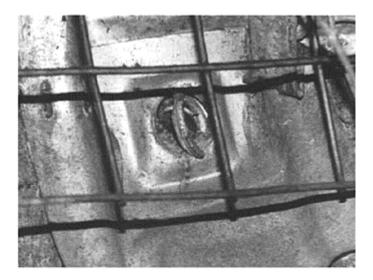


Figure 5. Damage of a reinforcing ring of the friction stabiliser due to wrong installation technique.

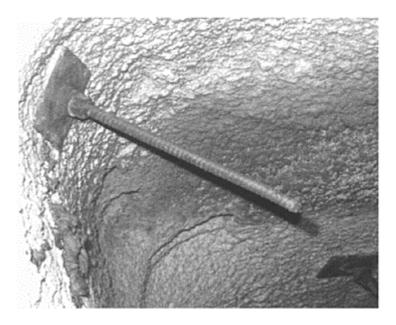


Figure 6. Protruding bolt—lack of a quality control system.

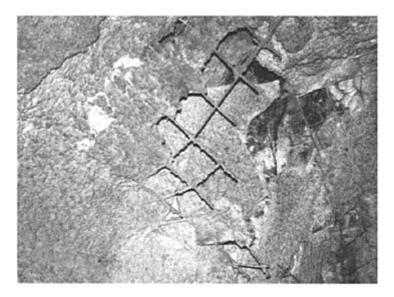


Figure 7. Inconsistent application of shotcrete over welded mesh.

of grout used for bolting, cable bolting or shotcrete mix.

- Testing for mechanical properties of installed support e.g. Pull Out tests. A Pull Out test procedure should specify the number of bolts to be tested, method of recording, and should provide minimum standards for mechanical parameters that must be achieved.

Long term monitoring shall include observation of the interaction between the rock mass and installed support and observation for corrosion of steel elements. All instances of rehabilitation of areas supported in the past should be investigated and feedback provided for support design.

8.5 Instability of the rock mass

A procedure should be prepared on reporting and investigation of instability of the rock mass i.e. falls of rock and support failure. Standard report forms should be available. Information should include location, failure dimensions and mode, comment on stress change, description of geotechnical features, excavation and rock support details and results of monitoring (Department of Minerals and Energy, 1997).

9 QUALITY ASSURANCE IN GROUND SUPPORT PERFORMANCE ASSESSMENT

Ground support performance assessments serve to determine performance of installed support (e.g. deterioration due to corrosion, failure due to mining induced stress or mechanical damage). It allows locating any potential uncontrolled instability of ground before the ground becomes unstable and hazardous. Early detection of failure allows mine operators to plan and implement actions limiting the effects of impending failure. Geotechnical monitoring is also carried out to assess changes in rock mass behaviour in time. It may include taking readings of geotechnical instrumentation and making periodical observations. Ground support performance assessment is done by inspection, monitoring and instrumentation.

9.1 Inspection

A competent person should be designated to carry out a geotechnical inspection of all areas affected by mining operations. A geotechnical inspection should be undertaken, on a regular basis, to check whether working activities or a work place comply with requirements written down in work procedures, specifications, practices or standards. All changes in ground conditions and geotechnical warning signs of impending instability should be reported. It is recommended that a geotechnical inspection checklist is used and results of each inspection are written down in an inspection record book.

9.2 Monitoring of ground support performance

During mining operations, a system of ground support performance monitoring and reassessment of mine design should be undertaken. The monitoring program should be specified in the Ground Support Management Plan. The plan should specify type of monitoring (observations and data recording), its frequency, type for interpretation, specify alarm trigger values for impending failure. The frequency of inspections should be relative to the risk and must take into account changes in ground and operating conditions. Performance of the rock mass should be closely monitored by competent persons. Results obtained through ground support monitoring should serve to refine the support design process. All substantial changes of the monitored values have to be communicated to designated employees. Areas of potential instability should be carried out for areas of high risk and a permit system to work in such areas is required. Monitoring results and recommendations following from their review should be passed on to relevant authorised personnel at regular intervals.

The information and data should be collected according to a defined standard. Geotechnical information and data should be collected on rock mass, status of excavation and support conditions. The following are examples how the data could be classified:

Rock mass conditions:

- Sound ground,
- Structurally damaged,
- Stress damaged,
- Weathering rocks,
- Wet conditions.

Rock mass classification:

- Very poor,
- Poor,

- Good,
- Very good,
- Extremely good.

Rock mass/pillar status:

- Stable,
- Minor deterioration,
- Some deterioration,
- Severe deterioration,
- Unsafe.

Mode of rock mass failure around excavations:

- Fall of ground,
- Convergence (creep, swell, closure),
- Crack propagation (spalling, fretting, buckling),
- Shear movement along structural planes,
- Seismic damage (strain bursts, rock bursts or gas outbursts).

Location of failure on the contour of excavations:

- In the back,
- In the shoulders,
- In the ribs,
- Floor heave,
- Other.

Size of potentially unstable blocks:

- Larger than 1 m in any direction,
- From 0.3 m to 1 m in any direction,
- Smaller than 0.3 m in any direction.

Support conditions:

- Not supported,
- Installed improperly,
- Damaged,
- Satisfactory,
- Good.

Type of damage to support:

- No damage,
- Mechanically,
- Corroded,
- Stress damage,
- Other.

Additional information should include comments on geology, the future use, and evaluation of risk of ground deterioration. The review should identify excavations where

inadequate support was installed, ground deterioration took place (e.g. due to stress or weathering), or ground support is not effective (e.g. due to corrosion).

Geotechnical personnel, after the inspection and consultation with the other team members, may prepare rehabilitation recommendations on immediate support—what is required to make the area safe immediately—or on permanent support—what is required to make the area stable for its expected life.

Field observation can be used to identify areas of a mine that are being consistently over or under-supported, or where no technical reasons are being used to install a particular type of support.

As a result of the support review program, for each designated area the following recommendations should be provided by specifying excavations which:

- Do not require any additional support work,

- Require periodical scaling of backs and ribs,
- Require support with bolts,
- Require support with bolts and weld mesh,
- Require cable bolting,
- Require permanent support with weld mesh and hotcreting,
- Require rehabilitation and permanent support.

9.3 Instrumentation

Geotechnical instrumentation, as determined in the Ground Support Management Plan, has to be effectively installed to fulfil monitoring objectives. Persons installing instrumentation should follow manufacturer specifications and readings should be taken, processed and interpreted on a regular basis by a competent person. For each piece of instrumentation, the value of early warning and alarm trigger has to be determined. An authorised person should monitor the results and all employees should be trained in an alarm system and an emergency procedure.

10 CONCLUSIONS

Safety and productivity in mines can be improved by implementing a quality assurance program in ground support management. The program is achieved by:

- defining of company policy and development of Ground Support Management System. The system should specify responsibilities and authority, document control system and competency and training required for each job.
- implementation of quality assurance in ground support planning and design. It should cover data collection and analysis, preparation and execution of Ground Control Management Plan, approval process and geotechnical feedback.
- introduction of quality criteria in work procedures and practices on drilling, barring down, and ground support installation.
- specifying quality factors in geotechnical inspections and monitoring.

Audits on quality assurance in ground support can provide mine management with the information on the status and potential for improvement in geotechnical activities.

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Support evaluation and quality assurance for AngloGold Ashanti Limited's SA region

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ABSTRACT: In January 2003 a new mining regulation came into effect, which requires that the employer must ensure that a quality assurance system is in place, which ensures that the support units are appropriate for the expected loading conditions. This places more stringent requirements on mining companies and by implication suggests a more structured approach to support evaluation and quality control of support products. A brief review of the industry indicated that very little work had been conducted on quality assurance with approaches differing between support manufacturers and mining companies. Within AngloGold Ashanti Limited the evaluation of new support products and quality assurance was generally conducted on an exception basis by individual mines and the Materials Engineering Department with the SA Region Standards Committee performing a co-ordinating role. This approach often resulted in inconsistencies in terms of procedures and technical requirements. This prompted the AngloGold Ashanti Limited South African region to embark on a programme to develop a consistent approach for the evaluation of support products and ongoing quality assurance.

1 INTRODUCTION

In January 2003 a new mining regulation (14.1) came into effect which states that: "At every underground mine where a risk of rockbursts, rock falls or roof falls exists, the employer must ensure that a quality assurance system is in place which ensures that the support units used on the mine provide the required performance characteristics for the loading conditions expected."

In response to this, the AngloGold Ashanti Limited South African (SA) Region embarked on a quality assurance programme on timber elongates through the Materials Engineering Department. It was soon apparent that this was not a straightforward issue with a wide range of opinions regarding testing methods, interpretation and rejection/acceptance criteria.

Concurrent to this initiative, the SA Region corporate Rock Engineering Department (RED) began to play a more involved role with the SA Region Standards Committee (SARSC) who is the controlling and co-ordinating body for product evaluation and quality assurance. It was noted that the various SA region mines tested many different

types of support but the approach was inconsistent and varied between mines. This could be considered a risk from a corporate governance perspective.

A brief review of the industry and local literature indicated that very little attention had been given to this topic and that the approach followed differed between companies. There also appeared to be confu-sion as to the promulgation of this regulation as the performance of support products was not generally regarded as problematic.

In 2003, AngloGold Ashanti Limited embarked on a strategic initiative of Commodity Strategy Development (CSD) with underground support as one of five main thrust areas. The purpose of the Underground Support CSD is to ultimately rationalise and optimise support systems and develop relationships with key suppliers that would ultimately improve safety and drive costs down.

From these various initiatives it was apparent that the evaluation of support products and the development of a quality assurance programme were interconnected and would form an integral part of the underground support CSD. This needed to be approached in a formal manner and be based on a sound technical base as well as being practical to implement.

2 DEVELOPMENT OF A SUPPORT TESTING AND QUALITY ASSURANCE SYSTEM

AngloGold Ashanti Limited SA region embarked on a programme to develop a protocol that would ensure a reasonable level of consistency and outline minimum requirements in terms of testing criteria and methods. Groundwork Consulting Pty Limited was contracted to assist with this initiative, which is managed and co-ordinated by the Corporate RED.

A decision was made by rock engineering management to initially focus on timber elongates as this is the highest risk support commodity by nature of the number of personnel exposed and the inherent variability of these products. The idea was to establish a detailed protocol for timber elongates which would also provide a framework for other support commodities.

An approach consisting of the following phases was adopted:

- An audit of current practices.
- Development of a generic procedure.
- Development of a testing and quality assurance programme for timber elongates.
- Implementation of the timber elongate programme.
- Expansion to other support commodities.

2.1 Audit of current practices for the evaluation and quality assurance of timber elongates

A comprehensive questionnaire was developed by Groundwork Consulting and circulated to the mines for completion by their Rock Engineering Departments. The purpose was to determine how timber elongates are evaluated before becoming standard stock items, what information was available and current quality assurance practices.

The quality assurance aspect was restricted to that conducted once the products had been delivered to the mines. This audit did not focus on quality assurance during the production phase as suppliers are expected to have a comprehensive quality assurance programme that is approved through the Materials Engineering Department.

The main findings of this audit follow: (Hayes & Piper, 2003):

- The original performance graphs and risk assessments for most of the products used or on standard stock are available on most mines.
- Most mines know their closure rate as it has been quantified at some point in the past. However, the distinction between closure rate and yield range did not come across clearly. The closure rate should determine the stiffness requirements and the length of time that the support will be working for.
- It was not altogether clear which support lengths were used in the original test graphs, and some graphs did not indicate this.
- None of the mines had creep test results. This is not an issue for mines with moderate to high closure rates, but may be required on mines where low closure rates occur.
- Most mines stated that they consider aspects such as stiffness and deformation range, when considering the initial test graphs. However, only one mine stated what criteria were used.
- There are procedures that can be followed when specific problems occur with the support quality. However, there does not seem to be a procedure in place to identify these problems up front before they manifest as failure of the unit underground.
- It is likely that the suppliers have quality control checks in place, but results of these are not given through to the RED on the mines as a standard procedure.
- An important concern, which was brought up, was that suppliers sometimes make changes to the product without the approval or consent from the relevant role players on the mines.

2.2 Development of a testing and quality assurance programme for timber elongates

Following the audit, Groundwork Consulting facilitated a workshop involving rock engineers from the different mines and Corporate Office. From discussions in this workshop it was clear that it would not be practical to adopt a pure statistical approach to testing as the time and cost requirements would be prohibitive.

The framework developed covers the following aspects:

- Product testing (laboratory and underground)
- Laboratory based quality assurance testing
- Underground quality assurance

Details of these various aspects are provided later in the paper.

3 SUPPORT EVALUATION AND QUALITY ASSURANCE PROCEDURE

Concurrent with the work being conducted on timber elongate requirements, a generic procedure was developed that outlined the general philosophy and requirements for all underground support classes. This is described below.

3.1 Objective of support testing

The purpose of carrying out a comprehensive testing programme is to determine whether the employment of the proposed new support system or modifications to the existing support system is appropriate for the anticipated loading conditions. The assumption is made that there is already an existing product on the mine and its performance is acceptable. For testing of any new product to be justified, the new product must be deemed to either:

- Have a superior specified performance to the existing product, and it must cost no more than the existing product.
- Have the same specified performance as the existing product, and it must be significantly cheaper than the existing product.

3.2 Roles and responsibilities

The initiation and motivation of support testing on a mine is the responsibility of the RED in consultation with the Production Manager. The RED is responsible for overseeing testing and making a final recommendation concerning a particular support product. The SARSC will review and ratify any recommendations.

3.3 Product review process

3.3.1 Product technical review

The RED will review the technical data presented pertaining to the proposed support product or modification to an existing product. The product will be compared to the applicable AngloGold specifications, guidelines and requirements. The support product will only be considered for underground testing if it has the potential to provide similar or better performance than the current support product (where applicable) and is also economically more viable and ergonomically practical. Reviewing of support products is the responsibility of the RED in consultation with the Production Manager.

3.3.2 OESH department review

The Occupational Environment Safety and Health (OESH) Department will review the Material Safety Data Sheet for the support product to ensure compliance with AngloGold Ashanti Limited standards. In the event of this data not being met, the supplier will be required to undertake further testing to ensure compliance. The OESH Department in

consultation with the RED will review the supplier's risk assessment to determine if it is satisfactory.

3.4 Documentation

Should the product review process described above be successful, underground testing of the product can be recommended. The findings and outcome of this review process are to be collated and documented by the RED. Guidelines and standard formats have been established for all documentation required from the suppliers and the motivating RED. The test application and supporting documentation is reviewed and ratified by the SARSC.

3.5 Laboratory testing requirements

This will be determined by the RED in accordance with AngloGold Ashanti Limited and Industry accepted specifications, guidelines and procedures. Where appropriate and practicable a statistical approach will be adopted to provide guidance on the number of tests required. All testing must be conducted at an impartial and accredited testing facility.

3.6 Underground evaluation

This will consist of two aspects:

- Firstly underground testing will be conducted on a limited scale.
- This will be followed by underground trials on an expanded scale.

The objective of underground testing is to obtain an understanding of the performance of a new support product in a particular environment and relative to current or existing support products/systems. Underground trials are conducted to assess the widespread use of the product.

The test site is to be selected by the RED in collaboration with the Production Manager and supplier. The test site must be representative of the general conditions under which the support product is intended for usage. Any risks associated with the testing of the support product must be identified beforehand, subsequently minimized and managed properly.

Testing will be conducted according to current mine standards unless otherwise specified by the RED in consultation with the Production Manager. Any deviation from the normal standards must be fully justified and formally recommended.

The supplier will be expected to provide adequate training in the use and application of their product. Training must be in accordance with AngloGold Ashanti Limited requirements.

Monitoring and instrumentation requirements will be specified by the RED in accordance with the applicable AngloGold Ashanti Limited specifications and guidelines. Instrumentation has generally been found to be extremely difficult and unreliable in an underground situation. Any instrumentation programme should be backed up visual observations, physical measurements and the recording of ground condition, safety and production data.

Minimum reporting requirements consisting of interim progress reports and a final test report are specified. The final report must include a recommendation on whether the support product is suitable for use and if it can be placed on standard stock.

3.7 Support quality control

Support quality control is necessary during the following phases:

- Manufacturing
- · Delivery and storage
- Underground installation

This paper does not discuss the manufacturing phase, as all AngloGold Ashanti Limited support suppliers must have an approved quality management system. This is evaluated and audited by the Materials Engineering Department.

The objective of a quality assurance system is to ensure that the support products (Piper, 2003):

- Satisfies the supplier's specifications and the performance benchmark established during the evaluation phase.
- Meets South African specifications and standards where appropriate.
- Is installed in accordance with the supplier's recommendations.
- Performs as expected in the underground environment.

3.7.1 Quality control of delivered and stored products

Quality control of products delivered to the mines will be conducted regularly. The frequency of quality assurance testing will vary depending on type of product and degree of variability. Any testing will be compared to the benchmark established during the support evaluation phase or the appropriate industry accepted standard or specification.

Quality control will also be conducted in the form of visual inspections of products and audits by Materials Engineering to ensure that products are properly stored and that effective stock management is practised.

3.7.2 Underground quality control

This is largely how support products are applied or installed and is the responsibility of line supervisors.

In some cases specialised quality control personnel will be used for payment purposes. Audits are conducted by the OESH and Rock Engineering departments to ascertain if support is applied or installed according to the appropriate standards. In addition visual monitoring of support also occurs to determine if support is performing as expected. This process allows sampling of large portions of the support population.

4 EVALUATION OF TIMBER ELONGATES

4.1 Testing of timber elongates

Three main objectives were identified for testing timber elongates (Piper, 2003):

- To ensure that the new product meets the required performance characteristics.
- To provide a baseline of product performance against which future quality control results can be compared.
- To compare the performance of the new product with that of the existing product in underground conditions representative of those where the product is likely to be used.

It is proposed that the performance characteristics of new timber-based elongates are quantified using a combination of laboratory testing, underground testing and underground trials.

4.1.1 Laboratory testing

This forms the first part of an evaluation of timber elongates. A substantial amount of work had been done on the testing of elongates by Daehnke et al. (1998) and Daehnke (2001) and this has been used as the basis.

Daehnke et al. (1998) suggests that a total of 27 units should be subjected to a variety of tests as follows:

- Five rapid displacement tests conducted at 3 m/s over a deformation range of 200 mm. A loading rate of 30 mm/min for the first 50 mm should be used prior to the initiation of the rapid displacement. Units must be tested to destruction or until 400 mm displacement has occurred.
- Ten slow tests at a loading rate of 30 mm/min. Units must be tested to destruction or a minimum of 400 mm displacement.
- Three slow tests at a loading rate of 30 mm/min on a 10 degrees grooved platen. Units must be tested to destruction or a minimum of 400 mm displacement.
- Two creep tests. Units with pre-stressing devices will be set at 200 kN. Units with no pre-stressing device must be set at 80 kN. Units must be loaded by the initial compression and their load shedding monitored over 7 days.
- Two slow tests at a loading rate of 10 mm/day for a period of 7 days.
- Five underground tests making used of suitable load cells and convergence measurement devices.
- No creep tests need be performed where closure rates are in excess of 2 mm/day.
- No rapid tests are required where units are not designed for use in seismically active areas.

Table 1. Probability of exceeding support performance specification (Daehnke et al., 1998).

	90%	95%	99%	
n=1	x=μ-1.282σ	x=μ-1.645σ	x=μ-2.326σ	
n=3	x=µ-0.740σ	x=µ-0.950σ	x=μ-1.343σ	
n=10	x=µ-0.405σ	x=µ-0.520σ	x=μ-0.736σ	

Due to the inherent variability of timber elongates Daehnke (2001) recommended the use of support performance design curves that ensure a high probability of exceeding support performance (90% or 95% confidence). This can be determined by the following relationship.

 $x=\mu-\alpha\sigma$ when n=1,

(1)

where x=sample load; μ =mean load; σ =standard deviation; n=number of interacting units; and α =a factor dependent on the probability of exceeding the sample performance and sample size.

The degree of interaction between support units must be decided upon. This will give the values of n, which in most cases in deep or intermediate mines should be 1. In shallow mines where support is more likely to interact, n could be given a value of anything between 2 and 5. Where possible, actual performance curves should be used rather than load correction factors, since the load correction factors vary widely with support type.

Table 1 should be used to determine the lower cutoff of exceeding support performance based on a particular confidence level and interaction of support units. Note that the confidence levels are only applicable to the lower limit and 95% implies that we are 95% confident that the support will perform above the values shown on the load-displacement graph.

Suppliers are expected to test according to the above procedure and supply the raw data and composite graphs from an accredited and independent facility. In the case of the supplier using their own test facilities the graphs must be accompanied with an independent appraisal. Figures 1 and 2 are examples of test results.

In addition to load-displacement graphs, photographs are required at different stages of deformation during testing. This allows for an impression of failure modes and can be used to compare and calibrate underground performance in the absence of instrumentation.

4.1.2 Underground testing

The main purpose of underground testing is to obtain an understanding of the performance of the elongate, in a representative underground environment (Piper,

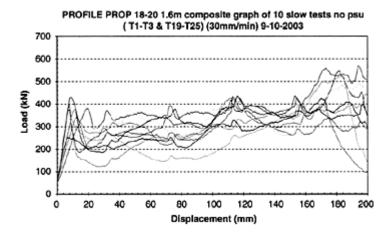


Figure 1. Load-displacement graph for ten profile props under slow testing conditions.

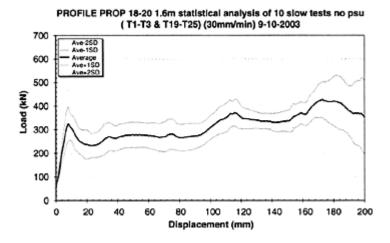


Figure 2. Load-displacement graph showing the mean and both one and two standard deviations under slow testing conditions.

2003). The emphasis is on using the underground environment to provide conditions that cannot be reproduced easily in the laboratory. The underground testing also provides an opportunity to evaluate nonperformance related factors such as ease of transportation, ease of installation and worker acceptability.

The main differences between laboratory and underground testing are as follows:

- Exposure to blast and scraper damage and its effect on performance.
- Condition of hanging wall and footwall surfaces in contact with the elongate and their effect of modes of deformation.
- Much slower rates of deformation than practical in the laboratory.
- Possibility of seismic activity and higher rates of deformation.

As many variables as possible should be eliminated at this stage of elongate testing. Good quality of installation is crucial. Instrumentation should be included in this phase as well as detailed observations of ground conditions and visual evidence of the performance over time. It is useful to compare performance against existing products that are already being used.

4.1.3 Underground trials

Following a successful underground test this would be expanded to underground trials on a progressive basis. Essentially, the test is expanded to several panels, then to a stope and eventually over several stopes or possibly mine wide. The idea is to evaluate performance over time under a variety of conditions. Following a successful trial period the product could be placed on standard stock. This would result in a contractual obligation from the supplier in terms of ongoing quality assurance.

5 TIMBER ELONGATE QUALITY ASSURANCE

Once a new product has been successfully tested it can be placed on Standard Stock. After placement on to Standard Stock it is essential that quality assurance testing is conducted on a regular basis. The main objective of quality assurance testing is to ensure that the performance of the product supplied remains within the performance requirements of the customer, as determined from initial benchmark testing or support evaluation phase. The supplier, who will bear the cost and be expected to contract in an independent reviewer, will conduct this testing. The following quality assurance testing guidelines are proposed (Piper, 2003).

5.1 *Physical characteristics*

Assessment of the physical characteristics of a product can provide an important first step in estimating the performance characteristics. This would include the physical dimension of the timber elongate and other physical characteristics, such as cracks in the timber. In the case of timer elongates these requirements are specified in AngloGold Specification 271/1 Issue 1.

This specification forms the basis of acceptance or rejection of products by means of physical inspection without the need for destructive testing. The advantage of this type of quality assurance is that a large percentage of the total products supplied can be evaluated.

In addition to physical checking of timber elongates on delivery it is necessary to implement effective stock control measures to ensure that the timber elongate quality does not deteriorate with time and exposure to the elements. This deterioration in performance is well documented. It is envisaged that the supplier will provide a certificate indicating the shelf life of a product that will be used to determine stock management controls.

5.2 Laboratory testing

Due to the variability of timber elongates for reasons that are not apparent in their physical characteristics it is necessary to conduct laboratory testing on samples of the products delivered. The purpose of these tests is to establish whether or not the product performance characteristics conform to those required by the customer. It is not practicably possible to test a sample that would satisfy traditional statistical sampling criteria. The following testing procedure is used. This is based on the testing procedure outlined by Daehnke et al (1998).

5.2.1 Type of testing

The following is required:

- Testing must be conducted at an AngloGold Ashanti Limited accredited testing facility.
- Testing should be conducted at a height of 1.6 m unless otherwise specified.
- Compression tests at a slow deformation rate 20–30 mm/min allowing for variability in deformation rates from one press to another.
- Rapid tests are usually 3 m/s but a cyclical test is recommended with repeated cycles of 50 mm at 20–30 mm/min followed by 200 mm at 3 m/s, until the elongate fails or the press capabilities are exceeded.
- Five slow and five rapid tests are required unless the product is not used in seismic conditions in which case ten slow tests are required. The five rapid tests also provide some information on the slow loading performance.
- If the product fails, another five for each type of test must be conducted.
- Include suppliers recommended pre-stressing unit (PSU) or the customers preference.

5.2.2 Frequency of quality assurance tests

Maximum interval of three months per product, unless otherwise agreed. This will enable seasonal variations to be assessed whilst minimising testing costs. In cases where a problem has been identified from underground observations, additional testing will be conducted immediately.

5.2.3 Sampling

As many of the mines would be using products delivered from the same source, samples will be taken from each mine storage area on a rotational basis. Samples will be selected by the mines RED. The products in the worst condition but which are still within specification will be selected.

It is not always possible to establish the age of the product. The products with the worst visual condition (cracking or greyness) are likely to be the oldest and possibly perform worst. Products, which are out of specification visually, should be rejected.

If the worst visual condition sample is inside the performance specification then the remainder of stored product should be. If the product fails then the bundle in the next worst visual condition should be selected.

5.2.4 Rejection of timber elongates

In terms of the volumes used and rate of consumption it is difficult to reject batches due to the short turn around time. If the performance of any product tested is outside limits specified by the customer the entire batch could be rejected. The normal practice would be to increase the sample size and investigate reasons for the poor performance.

5.2.5 Documentation and analysis of results

Photographs of all the samples tested are required. In addition photographs are required at the start of testing and at deformation intervals of every 50 mm until the product fails or the press range is exceeded. These photographs can be used for comparison with actual underground deformation characteristics and as a benchmark.

The load-displacement graphs listed below should be calculated from the test results, using common data points from each individual test graph at intervals of deformation of not greater than 1 mm. This should be done separately for each batch of slow tests and each batch of slow/rapid cycle tests. The calculations assume the support system consists of one unit rather than as a system of more than one unit.

- Mean of all the tests samples.
- Plus and minus one standard deviations. From this data the required confidence level can be calculated.

The following is required in the quality assurance test report:

- A clear statement on whether the performance of the products are within or outside the performance requirements and whether or not the batch from which the samples were taken should be accepted or rejected.
- All individual slow tests on one load-displacement graph.
- All individual slow/rapid cycle tests on one graph (if applicable).
- Composite load-displacement graphs indicating the mean and ± one standard deviation for slow and slow/rapid cycle tests.
- A table of key testing information, including product name, nominal size, test date, test height, deformation rate, test facility, test supervisor, source of product, orientation of product, and any other relevant information.
- Table of key product dimensions for each product tested, including any deviations from specifications.
- \bullet Table summarising key test results for each product as well as mean and \pm one standard deviation.
- Appendices detailing product specifications and test procedure.

- A signed statement from the supplier of the products tested confirming that the results from this testing are representative of the specified product and of the product being supplied to the customer.
- A signed statement from an independent technical auditor who has supervised and reviewed the results.

5.3 Underground assessment

Timber elongates exhibit a wide variability in their performance characteristics. The extent of this variability is such that statistically derived sample sizes are large. For example, the variability of a typical mine pole is such that as many as 100 tests would be required to satisfy a 95% statistical confidence level for each product (Piper, 2003).

It can be argued that rock falls are very seldom caused by failure of timber elongates. Therefore, it is unnecessary to conduct this level of destructive testing. The main purpose of testing is to understand the variability of the support units and to ensure that their performance characteristics are within the requirements specified by the customer.

However, the use of a large quantity of elongates in stoping operations is the one opportunity where the sample size requirements for testing can be met. All elongates used in the mine are essentially compressed a greater or lesser degree by the converging rock mass which enables at least their visual performance to be monitored.

If an understanding of the relationship between the visual performance and the performance characteristics (load-displacement) has been derived, the visual performance can be used as an assessment of the performance characteristics. This is proposed as the basis of the underground quality assurance programme.

It is essential that the different modes of deformation are determined from underground observations and these modes of deformation are reproduced in the laboratory to quantify the load-displacement characteristics associated with each. Using this information and the observed mode of deformation underground, the underground performance characteristics can be estimated for a large number of elongates.

It is suggested that all elongates in use are evaluated on an exception basis. This means that only those products that are not performing to expectations are recorded. This type of monitoring will require all products to be observed on a regular basis.

Supervisors and rock engineering auditors perform this monitoring. The focus is on application of the products in terms of quality of installation and adherence to standards as well as noting unexpected modes of deformation or problems.

6 DISCUSSION

Development of the generic procedure for the evaluation and quality assurance of support products and the work done on timber elongates has provided a framework for the development of procedures and guidelines for other support types. This work will be expanded to the following areas:

- Timber packs
- · Cementitious packs
- Tendons

- Grouts
- Shotcrete
- Pre-stressing devices
- Backfill
- Fabric support

Individual procedures will be based on AngloGold Ashanti Limited, industry and South African Bureau of Standards (SABS) specifications and guidelines where appropriate. Cognisance will also be taken of industry research and international practices.

The underlying philosophy is to develop practical and consistent procedures that have a sound technical base. In cases where standards or guidelines have not been developed for particular products this will be done by involving the suppliers, rock engineers and the end users. Problematic issues such as rejection criteria will be solved in a consultative manner through workshops and consultation across the industry.

These initiatives will assist in improving understanding of the factors that influence the performance of different support types.

7 CONCLUSIONS

The evaluation of new support types and ongoing quality assurance and control cannot be separated as the former provides the benchmark against which quality control will be conducted. A holistic approach is required that considers both laboratory and underground aspects.

Support testing and quality assurance procedures and protocols will ensure that the SA Region mines of AngloGold Ashanti Limited adopt a consistent approach and ensure minimum acceptable requirements.

The successful implementation of these initiatives will depend on co-operation between the suppliers and the end user.

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Ground support practices at Brunswick Mine, NB, Canada

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ABSTRACT: Brunswick Mine is located in the province of New Brunswick, Canada. This wholly owned property of Noranda Inc. produces 10,000 tons per day of zinc and lead ore at an average grade of 8.75%. This technical note contains a summary description of products, techniques and equipment pertaining to ground support practices at Brunswick Mine. It aims at depicting working methods and some of the typical reconditioning and development support applications.

1 THE BRUNSWICK MINE

1.1 Location

The Brunswick mine, wholly owned by Noranda Inc., is an underground operation producing a nominal quantity of 10,000 tons of ore per day from a polymetallic orebody containing zinc, lead, copper and silver. The mine, in continuous operation since 1964, is located near the city of Bathurst, New Brunswick, Canada (Figure 1). The main extraction method is long-hole stoping. Two shafts enable underground access, namely Shafts #2 and #3, whose sinking depths are respectively 900 and 1375 m. More than 100 million tonnes of ore have been extracted so far.

1.2 Background

The high extraction rate, the complex geology and adverse stresses in some areas of the mine are some of the conditions that led to significant tunnel surface displacements. Ground support packages are adapted to these conditions wherever warranted. This is preferably done during the tunnel development process. This technical note describes various techniques that can help, in supplement to such tendon support practices, control tunnel deformation.

Brunswick's orebody is up to 1300 m in strike length and 200 meters in width. Mining zones consists of one to seven parallel massive sulphide stringers or lenses (Godin 1987). The bottom producing level is 1125 meters deep. The overall ore strike is nearly North-South and the average dip is 75 degrees West. The natural stress regime at the site is 1.9:1 in

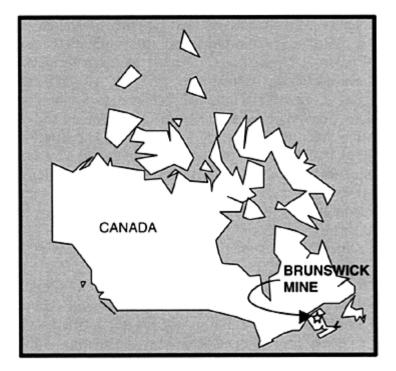


Figure 1. Brunswick Mine location map.

the E-W direction and 1.6:1 in the N-S direction (horizontal:vertical). Mining near the abutments of the orebody and into remnants generates high stresses.

More than 100 million tons of ore have been mined out since 1964. The mining methods over the years have been a mixture of surface mining and sub-level stoping, mechanized cut-and-fill, avoca and open stoping. The current mining method is long-hole stoping supplemented with paste backfill.

Section 2 contains a description of ground support practices at Brunswick.

2 GROUND SUPPORT PRACTICES

2.1 Products

Various tendon support systems are used at the mine. Table 1 contains a list of products used for tunneling and reconditioning. Tendon support characteristics are described as a function of steel type, nominal diameter, tendon length, yield strength and ultimate strength. Tendon strengths are these of the bars (as opposed to these of the threaded sections). Table 1 is presented for comparative purposes only.

2.2 Tendon support techniques

Brunswick Mine uses a variety of support packages depending on the expected level of stress in the mining area, the rock strength and quality. These support systems (Table 2) are preferably installed at the development stage. Headings are typically 5.0 m high and 5.5 m wide, apart from "shanty back" tunnels, driven along structural features and whose profile is generally larger.

All support systems indicated in Table 2 are used with 6 gage (4.1 mm) flat galvanized screen panels of 2.4 m×2.1 m size. Rockburst support is installed on top of 0 gage (7.6 mm) flat mesh straps of with the use of $0.3 \text{ m}\times2.1 \text{ m}$ size. Care must be taken to install the straps on top of the screen overlaps. All flat mesh squares are of 100 mm. All tendons use spherical seats. Tendon plates are 150 mm square domed and of 6 mm thickness for rock bolts, rebars and friction bolts. Plates are 150 mm square domed and of 9 mm thickness for MCB (Modified Cone Bolts) tendons. Mesh straps are installed lengthwise in the axis of the heading. Cable bolt plates are 200 mm square flat and of 9 mm thickness.

An illustration of the yielding cable bolt is provided in Figure 2, and one of the MCB in Figure 3. The yielding cable is a straight cable that is shaped with a nutcage bulb 0.3 m from its inner end. About 1.1 m of the cable, a third of the length, is fitted with a sleeve to prevent cement grout encapsulation of the cable in that section. The resulting "debonded" length is located in the middle of the cable. The MCB is essentially a smooth bar onto which is forged a conical shape with a resin mixing blade.

2.3 Shotcrete

Shotcrete usage for ground control at Brunswick Mine consists in the construction of reinforced shotcrete pillars and arches, the spraying of tunnel liners for paste and soft ground tunneling, and finally the spraying of walls for side-drilling (Gaudreau et al. 2003).

Reinforced arches are used in burst-prone and squeezing ground, for tunnel repairs, tunneling in soft ground and for brow support. The construction

Product	Steel type	Nom. diam. (mm)	Length (m)	Yield (kN)	Tensile (kN)
Rebar	20 M	19	2.2	125	188
Rock bolt	C1060	16	0.9	60	100
Friction bolt (galvanized)	_	39	2.2	(27–53)	89–124
MCB (greased)	C1055 M	19	2.2	117	169
Cable bolt (galvanized)	grade 270	16	7.6	_	284
Yielding cable bolt	grade 270	16	3.9	-	284

Table 1. Tendon support characteristics.

Support system	Back support	Pattern	Wall support	Pattern
Conventional	Rebar	1.4 m diamond	Friction bolts	1.0 m square
Conventional rockburst	MCB	0.9 m square and straps	Friction bolts	1.0 m square
Full rockburst	MCB	0.9 m square and straps	MCB	0.9 m square and straps
Deep squeezing	Yielding cables	1.8 m square and straps	Yielding cables	1.8 m square and straps

Table 2. Support systems.

guidelines are given in Table 3 and an illustration of a steel reinforcement set profile is provided in Figure 4. Profile members are pre-fabricated and assembled underground to fit the application requirements.

The arch pre-fabricated reinforcement set is built out of #3 rebar. Each set has a missing prong on top to permit easy overlap of the sets. The sets are secured in place using a 0 gage mesh strap and rock anchors. The full arch reinforcement shape is built by longitudinal juxtaposition of a number of sets and mesh straps. Securing the arch form with rock anchors, typically short friction anchors, reduces the amount of vibration while filling the reinforcement form with shotcrete. It will ensure a good bond with the steel.

Reinforced shotcrete arches can be installed side-by-side to augment the supporting surface when exposed to longer reaches. Table 3 describes the empirical rule practiced at the mine regarding the use of arches versus various drift spans. A space of 150 mm to 300 mm is left between the arch sets to avoid shooting directly through the mesh strap; it can cause segregation

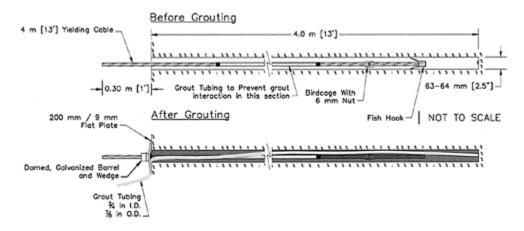


Figure 2. The yielding cable bolt.

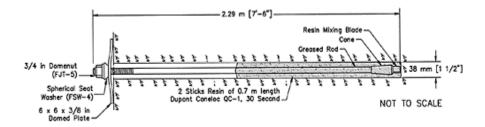


Figure 3. The Modified Cone Bolt.

Excavation span (in section)	Arch width (along tunnel axis)
<5.5 m	Single arch
5.5m≤span<7.5m	Double arch
7.5m≤span<9.5m	Triple arch

Table 3. Arch guidelines.

of the shotcrete. Arch profiles are typically spaced every 3 m in most cases but their spacing varies while proceeding to special tunneling practices in soft materials or while caring for a caved area. Arches are used in some drawpoints to control wall convergence or to control the caving of the stope brow. Arches can be combined to other construction practices such as reinforced shotcrete pillars for more versatility, especially when excavation spans are larger than 9.5 m (e.g. Table 3).

Reinforced shotcrete pillars are typically used for applications in large excavations subjected to static and dynamic loading. In some areas, the ground is sufficiently fractured above the large span that it becomes adequate to use "non-intrusive" methods to support the area, these methods involve the construction of reinforced pillars and/or arches.



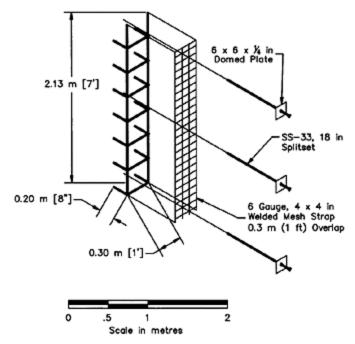


Figure 4. Shotcrete arch profile.

Reinforced shotcrete pillars are built using construction rings, expanded metal sheets and steel "waffles". Construction rings are made of a flat bar shaped in "C" with a rebar welded at its extremities. The construction ring is used to attach and form the expanded metal sheets that will be used as a back-splash for the shotcrete spraying. The pillar construction is augmented with the use of reinforcing "waffles" made of #3 re-bar welded in a grid of 30 cm×30 cm inside a circular ring of 1.7 m diameter. These waffles are installed during the 1.8 m diameter pillar building process every 30 cm in such a way that they can offer additional tensional resistance through potential fracture planes in the pillar.

Other shotcrete applications for ground control are tunnel lining for refuge stations and for machine-wear control in ore pass access ways, and tunnel lining in soft or friable ground. Last, in all areas for which multiple blasts are planned in a long-hole side-ring drilling access way, the drilling face is covered with shotcrete to control damage near the holes due to high blast pressures. Long-hole side drilling in highly stressed ground is a significant component of the ground control philosophy at Brunswick Mine. A short illustrative study of this mining method is presented in section 4 as well as some examples of shotcrete usage.

2.4 Intersections

Intersections and excavations of large spans (>7m) are cable bolted on a 1.8 m square pattern. In burstprone conditions yielding cables are used with criss-cross mesh straps, or reinforced arches depending on the excavation geometry, the ground quality and its proximity to the mining front.

2.5 Soft ground

Spiling, or fore-poling is used for tunneling in soft ground such as rock fill. The tunneling procedure in rock fill is briefly described here. The tunnel entrance must first be supported at the sound rock brow with a reinforced arch. Then R38N self-drilling anchors of 4 m length are driven into the arch on the tunnel roof periphery at a +15 degree angle and at a spacing of 30 cm so that the collar of the poles can be supported by the arch. Rock fill is excavated once the fore-poling is completed at an advance of no more than 2 m from the arch as measured on the tunnel roof. A layer of 75 mm of fiber reinforced shotcrete is then spayed on the roof and walls and a small shotcrete berm is shot on the back near the face. The latter is used as a drilling surface for the next series of spiles. Once the spiles are drilled and installed, a full rein-forced arch set can be juxtaposed at the face through the protruding ends of the spiles. The arch is filled with shotcrete and an additional layer of 75 mm is sprayed on the back and walls of the heading for a total of 150 mm. The excavation process is repeated every 2 m. The spiles are installed in an arch shape.

Tunneling in paste fill does not require the use of fore-poling when the material exceeds a uniaxial compressive strength of 1 MPa. The excavation of the 5.5 m wide arch-shaped paste tunnels is done with an 8 yard (6 m^3) scoop for the core of the tunnel and a mechanized scaler for its periphery. The exposed paste is covered with a 75 mm of layer fiber reinforced shotcrete and 6 gage weld mesh installed with a pattern of 1.5 m friction anchors. Reinforced arches are installed on top of the shotcrete and mesh at a 3m interval and at a maximum distance of 6 m from the face.

3 MANPOWER AND EQUIPMENT

Table 4 summarizes manpower requirements for development and reconditioning. Table 5 contains a list of equipment requirements.

Table 4. Manpower for development and reconditioning.

64 miners	40 recon.	4 shifts of 10 men	
		3–5 men on shotcrete	
		(2 shooting—1 preparation or 4 shooting—1 preparation)	
		1–2 wall bolters	
		1 mechanized bolter operator	

		2 to 4 back bolters
2	4 dev.	4 shifts of 6 men
		1 jumbo man
		2 mechanized bolter operators
		1 blaster
		1 mucker
		1 preparation man

Table 5. Equipment requirements.

Equipment reconditioning	Equipment development			
1 air powered mechanized bolter	2 electric jumbos			
2 wall bolters (air jumbos)	3 electric mechanized bolters			
6 dry shotcrete pumps				
230 ton shotcrete bag carriers with booms				
2 encreters				
3 boom trucks				
8 scissor lifts dedicated to development and reconditioning				

4 ILLUSTRATIVE STUDY

Tunnels subjected to an induced stress field perpendicular to their axis can be susceptible to wall convergence. Figure 5 illustrates a footwall drive parallel to a secondary pillar where high stresses have dislocated the walls. These were reinforced with 9 gage flat screen panels, 2.1 m long 20 mm rebars, and a coat of 75 mm of plain shotcrete. The floor suffered from heaving and no signs of dislocation could be seen in the back, covered with the same ground support package and an additional pass of 7 m long twin cable bolts on a 1.8 m pattern. Note that this heading was closed and alternate mining plans were designated for the secondary pillar. The back eventually came down in 2003. This tunnel was driven in metasediments.

At Brunswick Mine, the metasediments have a strength of about 72 MPa (Labrie 1998) and massive sulphides including the ore zone have an average strength of 199 MPa (Kristof 1988, Simon & Gaudet 1998). In the core of the mine stope hangingwalls are mostly massive sulphides but the narrower extremities of the orebody and the hangingwall of the central area have metasedimentary hangingwalls and foot-wall drives are mostly in the same material.

The metasediments tend to squeeze-in when subjected to higher stresses whereas massive sulphides tend to fracture and spall slowly or violently Figure 6 illustrates a large stope overcut driven into massive sulphides. The walls suffered from stress induced fractures. This area was rehabilitated using reinforced shotcrete pillars to cut the span and transfer some of the vertical stress provided further relaxation of the area.

Figure 7 illustrates a tunnel driven about three decades ago in a secondary long-hole stoping area. The access presumably suffered from induced fractures and is also cut by a continuous fault striking 30 degrees off the tunnel axis and dipping at 75 degrees. The tunnel was originally rehabilitated using wooden



Figure 5. Example of tunnel will large sidewall displacement.

cribs. The tunnel size was recently enlarged to accommodate modern mining equipment using a combination of conventional support and reinforced arches as shown in Figure 8. This type of tunnel repairs whereby a certain static load from heavily fractured rock in the back and walls must be supported is normally done by increments of 2 to 3 m advance. Back and wall tendon support and mesh is installed over that maximum distance followed by the installation of a reinforced shotcrete arch.

Figure 9 shows a similar tunnel repair situation where a heavily fractured tunnel was repaired in a highly stressed area. The support systems used were a combination of full rockburst support and reinforced arches. The area later suffered a 2.4 local Richter magnitude event without significant damage requiring further repairs. The tendon support system and arch forms were installed using a mechanized bolter.

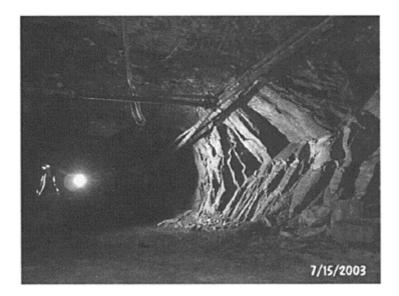


Figure 6. Example of excavation in massive sulphides under adverse stress conditions.



Figure 7. Tunnel repairs, 37 stope access prior to reconditioning.



Figure 8. Tunnel repairs, 37 stope access after reconditioning.

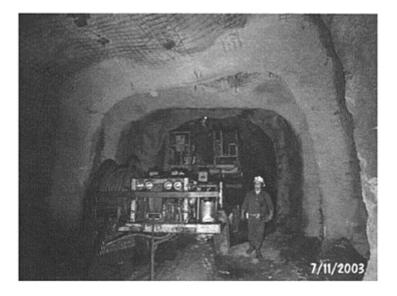


Figure 9. Tunnel repairs, rockburst support and reinforced arches.

Figures 10 and 11 illustrate the "Modified Primary-Secondary" mining method. This mining method permits mining of highly stressed sub-vertical ore without cutting drives into adverse stress conditions. At Brunswick Mine the ore is at least stronger than the country rock. Using conventional mining methods, it can be difficult to cut large stoping areas without loosing control of the hanging wall. The latter is in weaker metasediments for narrow mining zones.

The conventional Primary-Secondary mining method consists in down-drilling the primary blocks leaving secondary pillars of a set dimension, the size of which is mostly affected by the ability to drill across the stress flow. Production holes across the stress flow tend to squeeze and crush, making it difficult to recover the ore.

The Modified Primary-Secondary method consists in mining the primary stope using down-drilling and slashing the secondary stope into that opening. High stresses between the primary cut and the mined out front normally produces a significant amount of

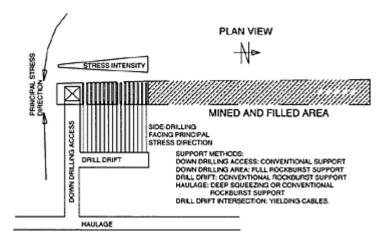


Figure 10. Mining method for highly stressed sub-vertical ore, Modified Primary-Secondary.

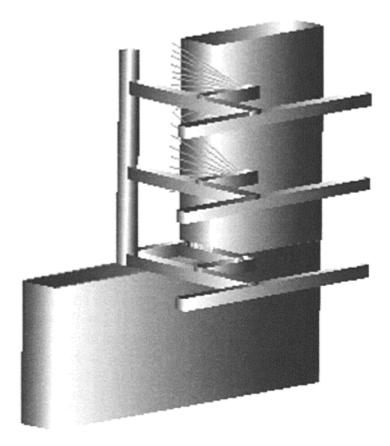


Figure 11. Isometric view of Modified Primary-Secondary mining method.

caved ore. The geometry of the mined out area and the position of the primary cut control the caving direction.

When headings are required close to the highly stressed mining front, such as for the access drive to the down hole drilling area and for draw points, the headings are driven facing the major principal stress direction. Headings located across the major principal stress direction are supported with conventional rockburst support or deep squeezing ground support. Such headings can be drill drifts for side-drilling or haulages for example. These headings are located approximately one to two stope spans away from the mining front mostly depending on the geometry of the ore lens and the side-drilling pattern and also the severity of the stress flow, at a distance from the mining front that will not affect their integrity when drilling as well as during the stope blasting process. Production holes for the secondary area are drilled facing the major principal stress at an incidence angle of no more than 40 degrees.

All highly stressed areas of the mine are side-drilled, i.e. drilled in the direction of the major principal stress. Driving tunnels in adverse stress conditions for production drilling can be avoided using this mining method. It has a significant impact on the quantity of rockburst support to be used as compared to other mining methods such as "mass blasting", where all production holes are drilled down and shot in one large blast.

5 CONCLUSION

Brunswick Mine had notable success with its support and repair methods for various ground conditions. The versatility of the support systems and the ease of installation when installed first-pass, combined with the technical knowledge of the crews, have greatly improved the safety and productivity of mining areas. Although further adjustments are required for mining and sequencing stope blocks under high stress conditions, it has been possible to drive new headings for these reserves into more favorable stress conditions thus requiring less ground support consumables. Techniques for driving headings in soft materials such as paste fill and rock fill have given additional versatility to the mining sequence.

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Civil engineering and tunnelling

100-year design life of rock bolts and shotcrete

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ABSTRACT: One of the main technical challenges of underground construction for public space is ensuring the long-term design life of support components. It is common for owners to specify a 100-year design life. Can designers, suppliers and constructors guarantee this? A few papers discussing this topic have been published over the past decade but as more underground public spaces are being built in Australia, the terms "permanent rock bolt" and "permanent shotcrete" have come under greater scrutiny. This paper presents the authors recent experience in relation to providing a permanent rock bolt and shotcrete support.

1 INTRODUCTION

The trend in tunnel design in Australia is to specify a 100 year design life for permanent support. Often that support is provided by rock bolts and shotcrete. Experience with concrete in compression extends back to Roman times but experience with the longevity of elements in tension is limited. For example, the British Standard BS8081 (Code of Practice for Ground Anchorages) has been available to guide the design of permanent rock bolts since the early 1990s.

This paper looks at the projects in Sydney, Australia undertaken since 1990 and the details of the installed rock bolts which are supposed to have equivalent design lives. The term rock bolt is used here in a generic sense and covers reinforcing elements comprising bars and strands.

2 DESIGN LIFE

2.1 Rockbolts

The life expectancy of rock bolts can be addressed from two viewpoints. The first is to attempt to assess the probable functional life of a given type of rock bolt in a given

hydrochemical environment. For example one may attempt to assess how long a Split-Set bolt may last in a particular tunnel given knowledge of the groundwater chemistry. This is the approach typically adopted by the soil anchor industry by incorporating an allowance for corrosion rate and including sacrificial steel (example NSW RTA's specification for the Design of Reinforced Soil Walls).

The second approach is to attempt to eliminate uncertainty by developing corrosion protection measures to provide a substantial level of safety. This is the approach presented in BS8081, which suggests permanent rock anchors require double corrosion protection of all components. The intent is that "*in the event of perforation of one of the two barriers during installation or loading, the remaining barrier must remain intact*" (Barley, 1997). In following this path it becomes readily apparent that attention to detail is critical (Pells & Bertuzzi, 1999). It is worth noting that BS8081 discounts the use of sacrificial steel stating it "gives no effective protection, as corrosion is rarely uniform and extends most rapidly and preferentially at localised pits or surface irregularities."

Table 1 provides a list of recent tunnelling projects in Sydney and the type and accepted design life of rock bolt support. A good engineering description of the rock mass conditions in Sydney is given in Pells, 2003 and Bertuzzi & Pells, 2002. All that can be said of the actual durability of the rock bolts used in these projects is that there have been no failures to date. Limited over-coring of rock bolts has been carried out and to the author's knowledge this has been done on temporary bolts.

Nevertheless, there appears to be an acceptance by the industry that carbon steel bolts cement grouted in an open ended plastic sheath are acceptable for 100-year design life in Sydney tunnels.

2.2 Shotcrete

Concrete technology is applicable to shotcrete. In the case of the Eastern Distributor a sacrificial thickness of shotcrete was required because of the local high acidity of the groundwater chemistry. No special treatment was required for the other projects in Sydney. However,

Project	Year	Permanent rock bolt	Design life	Reference
Opera house carpark	1990	Epoxy coated steel bolts fully cement grouted in 45 mm diameter holes	50	Pells et al, 1991
M2 tunnel	1993	Black steel 24 mm diameter bolts fully cement grouted in 44 mm diameter holes	100	Braybrook, 1993
Soil nail structures at Olympic park & Devlin st	1995	Open ended sheathed black steel M20 bolts fully cement grouted in 45 mm diameter holes (CT-Bolts)	100	Project design report
Wombarra drainage tunnel	1997	Black steel 24 mm diameter deformed bolts fully chemical resin encapsulation in 27 mm diameter holes	80 to 100	Project design report

Table 1. Permanent Rock Bolts Used in Sydney.

West Ryde drainage tunnel	1998	Black steel 24 mm diameter deformed bolts fully chemical resin encapsulation in 27 mm diameter	80 to 100	Project design report
Eastern distributor tunnel	1998	Epoxy coated steel bolts fully cement grouted in 45 mm diameter holes	50	Pells and Bertuzzi, 1999
		Closed ended sheathed multi-strand cable bolts fully cement grouted in 45 mm diameter holes (Freyssibolts)	50	
		Stainless steel bolts fully cement grouted in 45 mm diameter holes	75	
Bondi pumping chamber repair	1998	Stainless steel bolts fully cement grouted in 45 mm diameter holes	75	Project design report
Northside storage tunnel	1999	Fibreglass bolts fully resin encapsulated	100	Asche & Quigley, 1999
M5 East tunnel	2000	Open ended sheathed black steel M20 bolts fully cement grouted in 45 mm diameter holes (CT-Bolts), and open sheathed black steel cable bolts fully cement grouted in 50 mm diameter holes (Megabolts)	100	Adams et al, 2001
Cross city tunnel	2003	Partially closed ended sheathed black steel bolts with stainless head assembly fully cement grouted in 45 mm diameter holes (BBB-Bar)	100	Asche & Lechner, 2003
Epping to Chatswood rail link station caverns	2003	Specially designed open ended sheathed multi strand cable bolts (Megabolts) and single strand cable bolts (CT-Strand)	100	Project design report
		Open ended sheathed black steel, coarse threaded steel bar bolts with stainless head assembly fully cement grouted in 45 mm diameter holes (DCP & CT-Bolt)		

a complete water barrier may be required in ground-water environments more aggressive than Sydney, which gets us away from a rock reinforcement design to one of a passive lining. Shotcrete is not discussed further in this paper.

3 WHAT ARE THE PROBLEMS NOW

Some of the aspects currently being considered by tunnel designers are the details relating to the rock bolt head assemblies, temporary anchorage during grouting, rupture of plastic sheathing due to ground movement and final shotcrete cover. Of these the main issue in the author's recent experience is the potential for the plastic sheath to rupture when subjected to tension and shear loading. Design solutions typically offered are based on defining a maximum value for acceptable movement above which something must be done, including re-bolting, multiple stage grouting and the inclusion of a frangible or compressible grout. It goes without saying that none of these remedial measures are particularly attractive to the client or the contractor.

4 FAILURE MECHANISM OF ROCK BOLTS

Rock bolts typically fail in tension. It may well be that the start of the failure was shear movement but that typically leads to the rock bolt bending, necking and ultimately tensile failure. The failure involves composite paths: failure along the outer duct face over a proximal length translating to group strand failure and thence to multiple individual strand pull-out of the distal

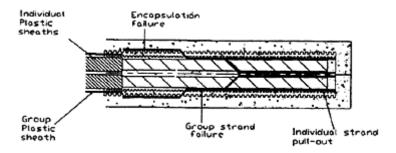


Figure 1. Encapsulation, group strand and individual strand failure interfaces (Barley, 2003).

component (Figure 1). This occurrence of progressive debonding is commonly accepted in the industry.

While the rock bolt itself fails in tension, its corrosion protection may fail much earlier in shear or actually puncture. A rock bolt can accommodate a relatively large amount of displacement, both axially and shear. A cable bolt typically can accommodate even more. However, if the object is to maintain the corrosion protection, then the amount of deformation that a bolt can be designed to withstand is that which ruptures the plastic sheathing. In other words, in many civil applications high capacity steel rock bolts are now being designed on the tolerance of the plastic sheath.

5 EXISITNG EXPERIENCE

5.1 Exhumed support

Weerasinghe & Anson (1997) investigated the condition of multiple strand cable anchorages after 22 years in a marine environment. The cables comprised greased and sheathed free lengths and cement grouted unsheathed fixed lengths. Interestingly, while there was evidence of general corrosion there was negligible loss of strand section within the single corrosion protection anchor. The main area affected by corrosion was that around and immediately beneath the anchor head that is in the detail where the greasefilled sheath connects to the stressing head and locking wedges. This case study suggests that that perhaps BS8081 is too restrictive in dismissing cement grout encapsulation as part of a corrosion protection system. The industry in Australia appears to be of the view that the cement grout does provide a layer of corrosion protection.

During 1997, excavation of a basement at No. 2 Bond St Sydney Steel intersected several steel strand cables that had been installed in 1972. The cables had been cement grouted in holes drilled through sandstone. These cables all showed a sign of corrosion and one was corroded. This case study suggests that cement grout alone does not provide long-term corrosion protection.

The European Code EN1537, which partly replaces BS8081, does allow cement grout to be considered to be part of the corrosion protection if it is within a plastic sheath and under working loads the cracks of the cement grout are less than 0.1 mm width.

5.2 Previous experiments

Barley (2003) describes results of the relatively limited testing of sheathed anchors subjected to shear that have been carried out carried out since the 1970s in the UK. His observations of the plastic sheaths, made after approximately 37 mm of shear, were that "the sharp edged grout fragments had severed and torn it (the sheath)." Barley further states that as a result of these tests, compliance with BS8081 has been restricted to axially loaded anchors, to wit while the concept of axial loading of curved stands and their corrosion protection components was recently tried for the Commonwealth Games Stadium in Manchester, UK, they were replaced with straight anchors during construction. In underground excavation, it is not possible to restrict permanent rock bolts to axial loads.

6 CURRENT EXPERIMENTS

6.1 Procedure

This author with his colleagues has carried out limited shear tests on grout encapsulated plastic ducts. Two series of tests were carried out.

The first series comprised grouting a 2 mm thick walled corrugated plastic duct within two hollow steel tubes. The two steel tubes were bolted together while the duct was grouted. After 7 days, one of the steel tubes was anchored to the concrete pavement and a

jack was used to push the second tube to simulated direct shear (refer to Figure 2). This author has also requested similar tests of bolt manufacturers. At the time of writing, some of the tests carried out by bolt manufacturer DSI were made available (Stevens, 2004).

The second series of tests was substantially more sophisticated and involved combined shear and axial loading of the corrugated plastic duct (refer to Figure 3). This series attempted to simulate the rock bolt within the rock mass. Two sandstone blocks separated by 5 mm of clay were placed into a loading frame. Smooth fiberglass strips were located on top of the bottom block to ensure the top block slide smoothly when pushed. A 65 mm diameter borehole was drilled through the blocks at 45° angle and a complete rock bolt (steel cable in this case and corrugated plastic duct) grouted into place. After 7 days, a jack was used to horizontally push the top sandstone block 15 to 18 mm whilst restraining vertical movement of the block. The relative movement of the two blocks was measured using crack monitor gauge installed on two sides. Following the test, the bolt was over-cored and inspected for damage. In one of the tests the horizontal movement was incrementally advanced; the test taking a week to reach the 18 mm of horizontal movement.

6.2 Results

The data suggests that the tested corrugated plastic ducts are damaged at approximately 15 mm of shear

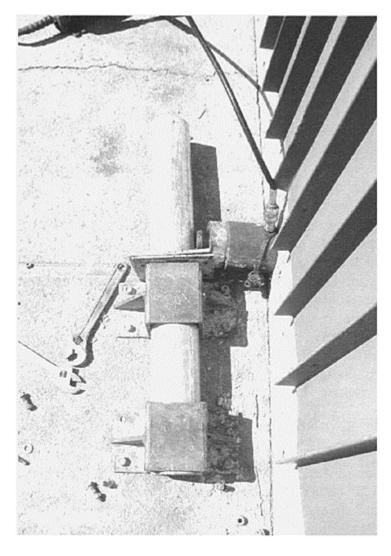


Figure 2. General layout of the apparatus in the first series of tests.



Figure 3. General layout of the apparatus in the second series of tests.

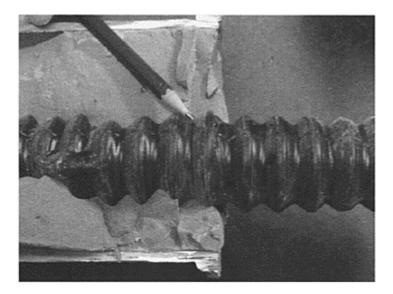


Figure 4. Close-up of damaged plastic duct after 15 mm of direct shear movement.

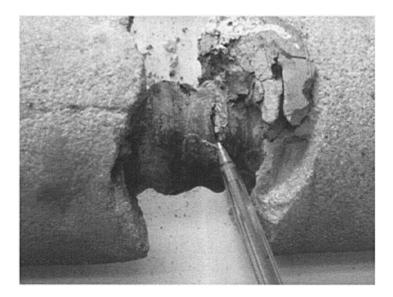


Figure 5. Close-up of damaged plastic duct subjected to combined shear and axial movement (after 18 mm of shear).

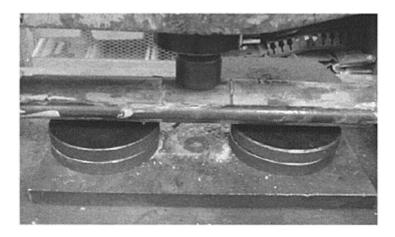


Figure 6. DSI's rig for direct shear test.

movement. The damage was consistent in all tests being caused by sharp fragments of broken grout puncturing the plastic duct. In the tests, little or no local failure of the sandstone around the bolthole occurred.

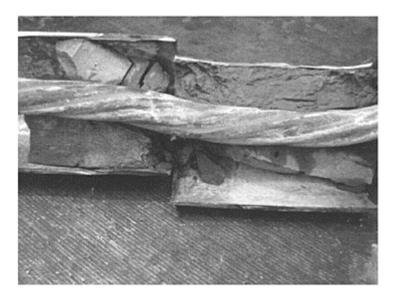


Figure 7. DSI's epoxy coated strand subjected to 23 mm of direct shear movement.

The tests carried out by DSI on their epoxy-coated cable, which were similar to the first series, suggest that this product is not damaged until about 20 mm of shear movement.

It is acknowledged that the first series of tests are simplistic because they do not represent the crushing of the rock; the dilatancy of the joint plane or the local debonding of the rock bolt; and hence may be overly aggressive. These shortcomings were partly addressed in the second series of tests. It is expected that the second series closely resembles the real case, although the test frame was too light to assess the influence of bolt pre-tension.

7 CONCLUSIONS

There appears to have been an acceptance by the industry that (i) cement grouts alone do not provide long term corrosion protection for carbon steel; and (ii) carbon steel bolts cement grouted in a plastic sheath is acceptable for 100 year design life in Sydney tunnels. However, in order to maintain the corrosion protection, the amount of deformation that a bolt can be designed to withstand is that which ruptures the plastic sheathing. In many civil applications high capacity steel rock bolts are now being designed on the tolerance of the plastic sheath.

This author has reviewed the available data and has carried out limited shear tests on grout encapsulated plastic ducts. The data from these tests suggests that plastic ducts are punctured by sharp fragments of broken grout at approximately 15 mm of shear movement. DSI carried out basic tests on their epoxy coated cable which suggest that this

product is not damaged until about 20 mm. The testing frame used was too light to assess the influence of bolt pre-tension however, further test work is continuing.

ACKNOWLEDGEMENTS

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Design and construction of water dams against 1000 m hydraulic pressure

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ABSTRACT: In the Ruhr basin it became necessary to separate neighbouring mines hydrologically from each other. This is possible only, if defined waterways do exist, which are accessible and can be safely closed. In the specific case, two mines had a connecting-drive of about 35 m^2 cross-section at a depth of 1000 m. The different aspects of the problem had to be handled in a multidisciplinarily fashion by experts in mining, geotechnics, civil engineering, and building-materials-technology. The safety of the construction was proved by methods used in civil engineering for not reinforced concrete structures. The material needed is a concrete according to standards in civil engineering. More than 1200 m³ fresh mix were produced underground and pumped into the site as a selfcompacting concrete within 48 hours. Following the hardening of the concrete the dam frustum was connected with the rock by cement injection into the joint.

1 INTRODUCTION

To separate the mines Heinrich Robert and Königsborn in the Ruhr basin hydrologically from each other, a dam in a depth of 1000 m was the favourable technical solution. The southern mine Königsborn had a water inflow of about 4 m³/min and was going to be abandoned, the northern mine Heinrich Robert was going to continue work with a manpower of about 4000 miners.

With a cross-section of about 35 m² and a water head of 10 MN/m² with a minimum lifetime of 25 years, the construction of a dam was not known up to now in carboniferous rock. The dam has been designed in the form of a frustum with the greater diameter of about 12 m faced to the waterfront. The length was determined for about 25 m including the two side-dams on both ends. Because of difficult access, the mixture of the concrete

had to be done underground. The components of the concrete consisted of slag furnace cement, aggregate of 8 mm maximum size, which had to be stored underground in "big bags" for some months.

After discussing the concept, the Deutsche Steinkohle AG (DSK) entrusted the Deutsche Montan Technologie GmbH (DMT), Essen, in cooperation with the Ingenieurgesellschaft Zerna, Köpper und Partner GmbH (ZKP), Bochum, with scientific consulting from Prof. Schorn, Technische Universitaet Dresden, with the planning and performance of quality management during the building phase (Hülsmann, Schlüter, Pieper, Schorn, 2002).

2 CONCEPT OF STRUCTURAL DESIGN

2.1 General

The two coal mines Heinrich Robert and Koenigsborn in the Ruhr Basin were connected by a drive of about 5 km in length and a crossection of about 35 m^2 at a depth of 1000 m. The mine Koenigsborn had been abandoned, but a flow of water of about $4 \text{ m}^3/\text{min}$ had to be pumped to the surface in order to keep away the water from the mine Heinrich Robert. It was expected that the water table could rise close to the surface, which meant that the dam had to be designed for a hydraulic pressure of 1000 m.

2.2 Foundation

The choice of the construction site had to meet the following requirements: an area of sufficient competent rock had to be found with sufficient low permeability

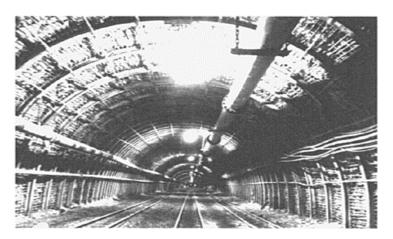


Figure 1. View into the open drive, going to be sealed.

against water with no influence of mining activity (subsidence) in the past or for the future.

The folded carboniferous rock (Westfalian A/ Namurian) consists of a series of layered sandstones and siltstones, which are differentiated by their varying content of quartz. Between these layers, coal seams occur in different distances.

Finally, a site was selected with a sequence of sandy siltstones with steeply inclined layers dipping about 70° towards the north. The cleavage planes showed normal distances of some centimetres up to about a decimetre.

Faults with dislocations up to 275 m were known, more than 1000 m from the actual site. Being potential waterways, the minor faults were classified as not water-leading, especially as there was no influence of mining activities for more than 2000 m in distance.

From driving the roadway in 1971 to 1974, it was known that the joints of the sandstone-layers were partially filled with water, which was dropping from the roof. This flow was diminishing with time so that it could be reasonably assumed that the water was stored in the rock and had no connection to water bearing layers.

2.3 Geometry of the dam

Because of the safety of some 4000 mine workers in the mine Heinrich Robert, the system of the dam construction had to be planned in a way that there could never be a failure. Therefore, the dam was designed as a double frustum, the longer one with an angle of about 6° against the axis of the cone, the shorter one with an angle of 45° . The longer frustum transfers the load of increasing water pressure into the rock, the shorter one decreases the stresses, especially tension, at the edges of the dam.

The two cylindrical side-dams with a length of 3 m each, also have the purpose of decreasing the peak

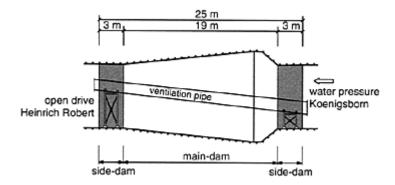


Figure 2. Longitudinal section of the dam including ventilation pipe.

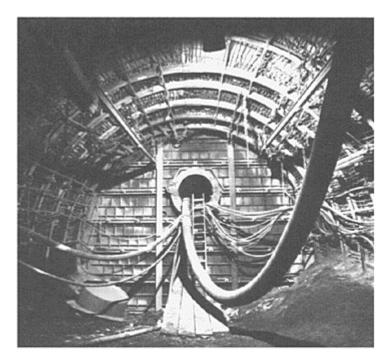


Figure 3. View at the face of the dam under construction from Heinrich Robert.

stresses at the ends of the main dam, but also have the more practical goal of providing a lining for pumping the concrete of the main dam. Furthermore, they give the chance of testing the planned operational work for the casting of the main dam, which has to be done fresh in fresh with 1200 m³ of concrete.

In order to obtain non loosened rock in the foundation mantle of the dam, the first 0.5 m of the mantle had to be excavated, and from here the above mentioned angles of the cone-shaped dam mantle were calculated. The largest diameter of the dam reaches 11.2 m, the smallest is 7.9 m; with the two side-dams the total length of the arrangement is 25 m. That means that the pressure of 10 MN/m^2 is efficient on 98 m².

In order to sustain ventilation, energy transfer, and transportation from one side of the dam to the other during construction time, a slightly conic formed pipe had to be installed. This pipe, eventually, had to be filled with concrete. Therefore, the larger diameter was on the water side, and the inclination of the pipe was larger than the angle of flow of the concrete.

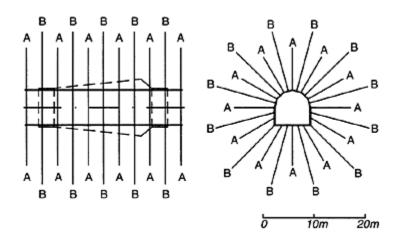


Figure 4. Drilling pattern for investigation and previous injection.

2.4 Primary rock improvement by injection

Prior to excavation, the foundation region had to be investigated with bore holes, which were also used for a grouting measure in order to stabilize the rock. The drilling pattern used was that at 11 rings with radial holes against the axis of the drive. Each ring had 12 holes. The holes in adjecent rings were rotated for 15° . The depth of the holes was alternating 12 m or 18 m, respectively, in each adjecent ring. The spacing of the rings was 3 m.

The cores gave information about the sequences of the layers, the intensity of cleavage, and the depth of loosened rock around the driveway. Furthermore, testing determined the uniaxial strength of the rock specimens to be approximately 60 N/mm², and the modulus of elasticity in the range from 30 000 to 70 000 N/mm². The coefficient of permeability was as low as 10 E–11 m/s.

In the boreholes, the rock was tested in-situ and showed a modulus of elasticity from 12000 to 16 000 N/mm2. Water-tests gave information about the coefficient of permeability in the range of $1 \times 10E - 7$ to $8 \times 10E - 10$ m/s.

The registration of pressure and volume of the suspension gave additional information about the state of the rock. The average input in the 12 m long holes was 5.9 kg/m applying a pressure <20 bar; the following injection of the 18 m long holes showed inputs of only 3.2 kg/m with pressures <30 bar. This confirmed the previous investigations.

With all these results the locality was determined to be free for excavation works.

2.5 Excavation

In the defined construction zone, the steel support had to be taken away and was, after extending the cross-section, replaced by anchors. This was done by a moderate blasting method to minimize ground disturbance. The anchor-plates had to be removed directly

before the concrete had to be brought in, in order to avoid difficulties, when drilling the bore holes for sealing the joint between dam and rock mass.

3 MATERIAL AND STRUCTURAL DESIGN

3.1 Requirements to concrete

As a result of a first and previous structural design, the necessity of a high quality concrete as used in civil engineering was recognized. Mortars or slurries as used underground do not have sufficient properties (Hülsmann, Schmidt-Schleicher, Schorn, 1984). A structural design according to standards for reinforced and none-reinforced concrete structures requires a material which is covered by the scope of the standards. Only concretes of relatively high compressive strength with cementitious binders and aggregates, according to the standards for structural design and execution were used. Even if these basic conditions are fulfilled, the safety concept of the standards in civil engineering can be transferred to dam structures for underground support.

The design concept was made according to the German Standard DIN 1045, which regulates the material properties and the structural design for concrete and for reinforced concrete structures. Cement and aggregates are chosen according to other national standards describing requirements for the components of those concretes which are covered by the scope of DIN 1045. The dam was planned and designed to the rules of none-reinforced concrete structures.

3.1.1 Compressive Strength

The compressive strength class to be required is class "B 35". That means a 5%-Quantile of 35 MN/m^2 as a minimum. The average compressive strength was found to be always above this value, e.g. 40 MN/m^2 . Compressive strength measurement is related to an age of 28 days after wet and dry storage. In the case of the water dam, the actual conditions are more favourable. The material keeps in wet state all the time. In the first weeks the evaporation of the dam takes took place with extremely low rates. The capillary pore system remains saturated with water all the time. Shrinkage does not take place.

The conditions for the hydration process of cement are excellent. The strength of concrete will increase after the 28th day. When the maximum water level loads the dam after some years, the strength will have increased up to about 50% more than at the 28th day, perhaps higher. Nevertheless the characteristic value of the compressive strength is taken from the 28-day-strength. The difference of strength acts as a hidden increase of safety.

Requirements for the concrete mix are not only determined by the right compressive class. In projects of great concrete masses the heat from the hydration process must be limited. A low heat slag furnace cement has been chosen. Additionally the cement content in the mix was minimized. But there exist other requirements contradictionary to the reduction of cement content. The mix has to be pumped to the site and placed with no possibility of using a tool for compaction. Only the falling fresh concrete out of the pipe produces a small amount of compaction energy. The material is quasi self compacting.

An increased cement or fly ash content is helpful in those cases. Instead of higher cement contents, highly efficient chemical additives were used. Those polymeric additives are produced for civil engineering purposes, their use in mining requires a special permit from the mining authorities.

The concrete mix could be pumped and was self compacting with a small amount of compaction pores which did not reduce the compressive strength to a lower compressive strength class than planned. The permeability of the capillary pores is not affected by the small amount of the round shaped and single situated compaction pores.

3.2 Execution

The production of fresh concrete had to be carried out continuously to avoid any joint in the dam. This condition concerns the requirements of the fresh concrete properties, as well as the execution procedures. Several properties were investigated, e.g. altering of flow over the time of processing and the increase of strength over the time of hardening. Even the influence of the temperature on the site above the temperature in laboratory of 20°C according to national standards were taken into account. The conformity and the quality control of the concrete were supervised by independent laboratories as required in the standards for civil engineering purposes.

The water in the coal mine contains sulphate. The concentration was determined by a chemical measurement. The values were compared to a list shown in the national Standard DIN 4030 and classified as lower aggressive. Slag furnace cement is able to resist the attack. Additionally, the main dam is protected against sulfuric attack by the side dam facing the water. Any damage would affect the side-dam only, not the dam.

The side-dams were built with the same material as the structural dam. That was not necessary with respect to the structural design. However, it was a very good occasion for testing the process of concreting and enabled special experiences to be geined. This was of great importance. Concreting underground on the same quality level as used in civil engineering is



Figure 5. Concrete mixing underground.

difficult, and it is not included in the experiences of most miners.

The total quantity of premixed aggregates and cement was stored underground near the site in "Big Bags" at 800 kg each. The material store fed 4 independent working mixers. The mixers had to produce a fresh concrete volume of $25.5 \text{ m}^3/\text{h}$ as an average. The number of mixers was greater than the calculated capacity due to redundance purposes. In case there was a failure of one of the mixers the others were able to continue concreting on the planned level. All equipment was inspected before the concreted process started to lower the risk of an early failure. After the side-dams were concreted the main-dam was produced by a continuous concreting process of 1200 m³.

4 INJECTION WORKS

After sufficient hardening of the concrete had taken place the structure had to be connected with the surrounding rock mantle through cement injection.

For this purpose, drill-holes were placed in a coneshaped mantle-surface, with different angles against the axis of the cone. These surface mantles were arranged in a

way that the joint between rock and dam was penetrated by different mantles. For demonstration, whether the joint actually was met, the transition zone had to be cored.

From the northern side, four cone-shaped mantles had to be arranged with 32 drillholes. The joint voids were filled with cement-suspension with low pressure of less than 5 bar.

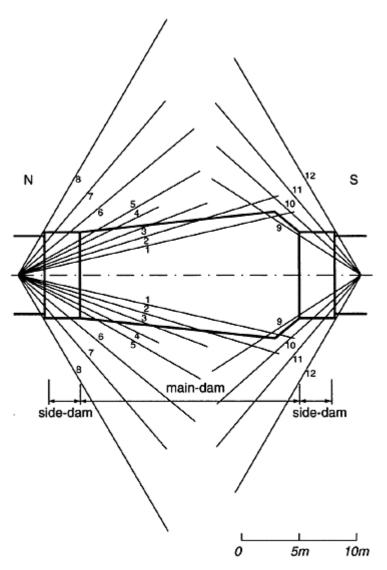


Figure 6. Drill cones for injection the joint between dam and rock and the rock mass.

In order to diminish the permeability of the rock and to prestress the dam with respect to its future load, from both ends of the dam drill-holes were located on four cone-shaped mantels with angels between 30° and 65°. All together 104 holes had to be drilled with a length of up to 20 m.

Beginning the injection from the ends and close to free surface, the pressure was kept low and did not exceed 5 bar. Leakages were observed between the holes, as well as in the joint between dam and rock. The pressure in the interior region was increased up to 90 bar. Finally, in the centre of the injection zone, where the pressure field approximated a three-dimensional status, the pressure raised up to 10 MN/m², an equivalent of the hydraulic pressure, which the dam would have to withstand.

Generally, a cement CEM III/B32,5 NWHS according (DIN EN 197) was applied, using a water/cement ratio of 1.0. To reduce the permeability, some of the injection-cones were treated with extremely fine cement Microcem A/SR. The average input of solid cement was relatively low with 5.7 kg per m bore hole. The water tests in the dam, the rock, and the

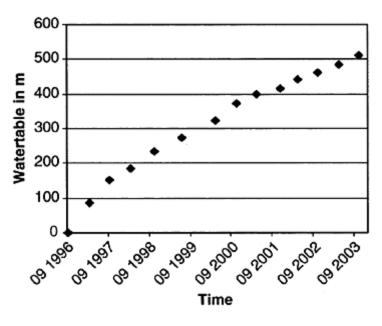


Figure 7. Rising water table behind the dam.

joint between dam and rock, showed a permeability of less than 1×10E-9 m/s.

5 CONCLUSION

The dam with an excavation cross-section of 98 m^2 had to be designed for a hydrological pressure of 10 MN/m². This extraordinary problem required the interdisciplinary

cooperation between the mine management, the mining and geotechnical consultants, the civil engineers, and building materials experts. A precise definition of the location for foundation and its investigation combined with a rock grouting ahead of excavation, a distinguished planning of the construction design with operational demands as well as applied building-materials technology development made the basis for a qualified construction of the dam. The connection of the dam with the rock by cement-injection, prestressing the joint and the rock, and filling the ventilation pipe with concrete made the function of the dam possible in 1995.

The water table still is rising and stands now more than 500 m above the dam level.

The construction of a water dam against high pressures today is possible according to standards of techniques and science. Depending on the amount of water inflow, the cost of construction will be met by saving pumping costs after a few years.

A critical value is the determination of the length of a dam. It certainly might be possible to calculate the stability of a shorter dam, but one has to bear in mind that a shorter dam allows the water to take a shorter way through the surrounding rock and might increase the water flow around the dam. Thus it is a delicate balance to find the optimal design for a competent and tight dam in each single case.

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Thermo-chemo-mechanical assessment of support effectiveness during tunneling in squeezing conditions

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ABSTRACT: Recently, the combination of shotcrete as the primary lining and of face reinforcement by means of fiber-glass dowels was successfully employed during tunnel excavation in squeezing conditions (e.g., in soft rocks under high overburden stress). In this paper, numerical results obtained from axisymmetric analyses considering the application of the shotcrete lining and the installation of fiber-glass dowels at the tunnel face are presented. In order to account for time-dependent processes in the ground, a viscoplastic material model was chosen. For the description of the mechanical behavior of shotcrete, a thermo-chemomechanical approach is employed. The obtained results provide insight into the ground-shotcrete interaction, the effect of face reinforcement by means of fiber-glass dowels, and the state of stress in the shotcrete lining. Moreover, the variation of ground properties allows evaluation of the loading and effectiveness of the employed support means for different geotechnical conditions.

1 INTRODUCTION AND DESCRIPTION OF THE PROBLEM

Large ground deformations characterize the excavation of tunnels in weak rock-masses, often resulting in unacceptable convergence, stability problems, and overstressing of the support elements. These conditions are generally referred to as squeezing conditions. Following the International Society of Rock Mechanics, "squeezing of rock is the time-dependent large deformation which occurs around the tunnel and is essentially associated with creep caused by exceeding a limiting shear stress. Deformation may terminate during construction or continue over a long period" (ISRM 1994).

The magnitude and rate of tunnel deformations related to squeezing depend on several factors such as the geological conditions, the *in situ* stress, and the groundwater regime as well as the geotechnical properties of the rock-mass and the tunneling technique. The

squeezing potential is mainly related to the f_c/σ_0 ratio between the uniaxial strength of the rock-mass and the *in situ* stress (Hoek 2001). Values of f_c/σ_0 less than 0.2 are found to be representative of very severe squeezing conditions.

The consequences of the rock-mass squeezing potential are strictly related to the adopted tunneling technique (ISRM 1994). Immediate support of the excavated tunnel reduces the yielding of the rockmass around the opening, thus limiting the short- and long-term deformations. Recently, the stability of the tunnel face was recognized as the critical factor for assuring the stability of the whole tunnel, especially in excavations characterized by spans larger than 10 m (Lunardi 2000, Hoek 2001).

Different solutions are adopted to overcome the difficulties related to face instability during tunneling in squeezing conditions. Sequential excavation methods (i.e., side-drift method, top-heading and benching-down excavation method) make use of smaller excavation sections to improve tunnel face stability. Full-face excavation methods (i.e., ADECO—RS method) offer the possibility of highly mechanizing the tunnel operations by using larger equipment (Schubert et al. 2000). To guarantee stability, face reinforcement by means of fiber-glass dowels is applied. This is followed by the placement of heavy temporary lining of shotcrete and steel sets as close as possible to the face.

This paper focuses on tunnels driven by full-face excavation in squeezing ground conditions, where the application of shotcrete and face reinforcement by fiber-glass dowels are instrumental in reducing ground deformations and ensure safe working conditions. Even if this construction method is widely adopted (Kovari et al. 2000), the available design methods are not completely satisfactory: tunnel design is usually performed empirically, on the basis of the experience of the contractors and consultants involved in the project.

In the remainder, numerical results obtained from 3D axisymmetric analyses of tunnel excavation are presented. The construction steps, including tunnel excavation, shotcrete application, and reinforcement of the tunnel face by fiber-glass dowels are considered in detail in the numerical model.

The time-dependent behavior of the rock-mass is described by the theory of viscoplasticity. For the description of the mechanical behavior of shotcrete, a thermochemo-mechanical model developed at Vienna University of Technology is adopted (Hellmich et al. 2001, Sercombe et al. 2000, Lackner et al. 2002).

The numerical analysis is performed in two separate steps. First, the thermo-chemical problem accounting for the thermally-activated nature of the hydration process of the shotcrete material is solved. This analysis provides the temperature field in the rock-mass and the temperature field and the hydration extent in the shotcrete lining. Then, the mechanical analysis gives insight into the ground-shotcrete interaction, the effect of face reinforcement by means of fiber-glass dowels, and the state of stress in the shotcrete lining. The numerical analysis aims at investigating the tunnel conditions both during the steady-state advancement of excavation and during excavation breaks, typically observed in the period of summer holidays.

Finally, the influence of the face reinforcement density on tunnel deformations is investigated in different geotechnical conditions by varying the rockmass cohesion in a wide range of typically-observed values.

2 CONSTITUTIVE MODELS

2.1 Viscoplastic material model for ground

A viscoplastic material model is employed to describe the behavior of rock masses. It allows for consideration of squeezing conditions during tunneling.

The plastic material response of the ground is described by means of the Drucker-Prager criterion (Fig. 1a) reading:

$$f_{DP}(\boldsymbol{\sigma},\boldsymbol{\zeta}_{DP}) = \sqrt{\mathbf{J}_2} + \alpha \mathbf{I}_1 - \boldsymbol{\zeta}_{DP} / \boldsymbol{\beta}$$
(1)

where ζ_{DP} represents the hardening force of the Drucker-Prager criterion. The parameters α and β are computed from the cohesion *c* and the friction angle ϕ such that the Drucker-Prager meridian coincides

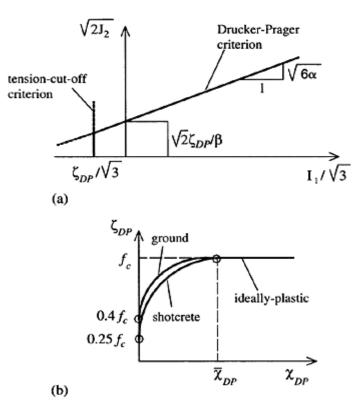


Figure 1. Multi-surface plasticity model for ground and shotcrete: (a) yield surfaces and (b) strain-hardening of Drucker-Prager criterion.

with the compression meridian of the respective Mohr Coulomb criterion. This yields:

$$\alpha(\varphi) = \frac{2\sin\varphi}{\sqrt{3}(3-\sin\varphi)}, \quad \beta(c,\varphi,f_c) = \frac{f_c}{\sqrt{3}c} \frac{3-\sin\varphi}{2\cos\varphi}$$
(2)

with
$$f_c(c,\varphi) = \frac{2c\cos\varphi}{1-\sin\varphi}$$

denoting the uniaxial compressive strength of the material. An increase of the hardening force ζ_{DP} from 0.4 f_c to f_c in the context of strain hardening is considered, see Figure 1b.

In the tensile loading regime, the tension-cut-off is employed, reading:

 $f_{TC}(\sigma, \zeta_{TC}) = I_1 - \zeta_{TC}$

(3)

with ζ_{TC} representing the hardening force. For the tension-cut-off, ideally-plastic behavior is assumed. Hence, $\zeta_{TC}=f_t=$ constant, where f_t is the uniaxial tensile strength, with $f_t\approx 0$.

The Drucker-Prager criterion and the tension-cutoff are combined in the context of multi-surface plasticity. The evolution equation for the plastic strain tensor is given by (Koiter 1960):

$$\dot{\boldsymbol{\epsilon}}^{P} = \dot{\boldsymbol{\gamma}}_{DP} \frac{\partial \boldsymbol{g}_{DP}}{\partial \boldsymbol{\sigma}} + \dot{\boldsymbol{\gamma}}_{TC} \frac{\partial \boldsymbol{f}_{TC}}{\partial \boldsymbol{\sigma}} \tag{4}$$

where $g_{DP}(\sigma) = \sqrt{J_2} + \overline{\alpha} I_{1\text{represents the plastic potential of the Drucker-Prager}$ criterion. $\overline{\alpha}$ is computed from the dilation angle $\psi_{\text{with}} = 2\sin\psi/\sqrt{3}/(3 - \sin\psi) \cdot I_{1}$ Equation (4), γ_{DP} and γ_{TC} stand for the plastic multipliers of the Drucker-Prager criterion and of tension-cut-off, respectively. The extension of the described multi-surface plasticity model, towards viscoplasticity, follows the law proposed by Duvaut & Lions (1972), reading:

$$\dot{\boldsymbol{\epsilon}}^{vp} = \frac{I}{\tau} \mathbf{C}^{-I} \left(\boldsymbol{\sigma} - \boldsymbol{\sigma}^{\infty} \right) \text{ and } \dot{\boldsymbol{\zeta}}_{DP} = -\frac{I}{\tau} \left(\boldsymbol{\zeta}_{DP} - \boldsymbol{\zeta}_{DP}^{\infty} \right)$$
(5)

where C denotes the elastic material tensor. In Equation (5), τ is the relaxation time, σ^{∞} and ζDP correspond to the solution for rate-independent elastoplasticity, i.e., to the solution of infinitely slow loading.

2.2 Thermo-chemo-mechanical material model for shotcrete

During the installation of the primary lining shotcrete is applied onto the newly excavated surfaces of the tunnel. Already at early ages, i.e., during the chemical reaction between

cement and water (hydration), the shotcrete is loaded mechanically by the inward moving rock mass.

For the simulation of shotcrete under such loading conditions, a thermo-chemomechanical material model was developed at Vienna University of Technology (see Hellmich et al. 2001, Sercombe et al. 2000, Lackner et al. 2002).

Shotcrete is modeled in the framework of chemically reactive porous media (Coussy 1995). The hydration process is described by the degree of hydration which is defined by the mass of hydrates formed, m, related to the mass of hydrates formed at complete hydration, m_{∞} :

$$\xi = \frac{m}{m_{\infty}} \quad \text{with } 0 \leq \xi \leq 1.$$
⁽⁶⁾

The thermally-activated nature of the hydration process is accounted for by an Arrheniustype evolution law for ξ reading (Ulm & Coussy 1995):

$$\dot{\xi} = \widetilde{A}(\xi) \exp\left(-E_a / RT\right) \tag{7}$$

where $\tilde{A}(\xi)$ represents the normalized chemical affinity. It is the driving force of the hydration process. E_a is the activation energy, R is the universal gas constant, with $E_a/R=4000$ K. T is the temperature in K.

Dissipation phenomena at the microlevel of the material are accounted for by means of internal state variables and energetically conjugated thermodynamics forces, related to the state variables via state equations. The rates of internal state variables are related to the corresponding thermodynamic forces by means of appropriate evolution equations.

The following dissipative phenomena govern the material behavior:

• the chemical reaction between water and cement, i.e., hydration, results in chemical

shrinkage strains ϵ , aging elasticity, strength growth (chemomechanical couplings), and in the release of the heat at hydration (thermo-chemical couplings); see Equation (6) already described;

- microcracking of the hydrates leads to plastic strains ϵ^{p} . The state of microstructural changes resulting from microcracking is described by the hardening variables χ_{as} in classical plasticity theory. In the present case, a multi-surface model consisting of a Drucker-Prager loading surface and a tension-cutoff loading surface is employed (Fig. 1);
- stress-induced dislocation-like processes within the hydrates result in flow (or long-term) creep strains ϵ^{j} . The state of the respective microstructural changes is described

term) creep strains \mathbf{e}^{\prime} . The state of the respective microstructural changes is described by the viscous flow;

• stress-induced microdiffusion of water in the capillary pores between the hydrates results in viscous (or short term) creep strains ϵ^{ν} .

During hydration of shotcrete, new hydrates are formed in a state which is free of microstress (Bažant 1979). This is reflected by an incremental stress-strain law, reading (Sercombe et al. 2000):

$$\Delta \boldsymbol{\sigma} = \mathbf{C}(\boldsymbol{\xi}) : \left[\Delta \boldsymbol{\epsilon} - \Delta \boldsymbol{\epsilon}^{p} - \Delta \boldsymbol{\epsilon}^{v} - \Delta \boldsymbol{\epsilon}^{f} - \Delta \boldsymbol{\epsilon}^{s} - \Delta \boldsymbol{\epsilon}^{T} \right]$$
⁽⁸⁾

where $\Delta \sigma$ represents the increment of the stress tensor, C the (aging) elastic material tensor and $\Delta \epsilon^{T}$ the increment of the thermal strain tensor.

In general, the different processes, i.e., thermal, chemical and mechanical processes, depend on each other. Such dependencies are referred to as couplings (Fig. 2). For example, the interaction between the hydration process and the deformations is denoted as chemo-mechanical coupling. The interaction between the deformations and the temperature is accounted for by the thermo-mechanical coupling. Finally, the interaction between the hydration and the temperature is referred to as thermo-chemical coupling.

On the basis of experimental evidence, some couplings have minor influence on the behavior of shotcrete (Fig. 3). For example, chemo-mechanical coupling resolves in a one-way coupling, with the hydration process influencing the mechanical state of the material through shrinkage strains, aging elasticity, and chemical hardening. On the contrary, mechanical deformations only exhibit little influence on the hydration process. In the same way, thermo-mechanical coupling reduces to a one-way coupling, with temperature changes resulting in temperature strains.

Based on the one-way couplings depicted in Figure 3, both thermal and chemical processes are not influenced by mechanical deformations. This permits splitting the numerical analysis into two parts: a

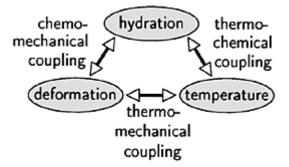


Figure 2. Material model for shotcrete: possible couplings between thermal, chemical, and mechanical processes.

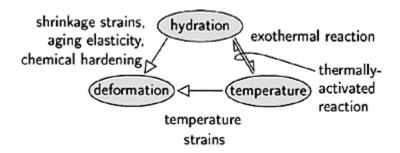


Figure 3. Material model for shotcrete: relevant couplings.

Table 1. Mixture characteristics and material properties of the shotcrete considered in the numerical analyses.

Cement content	$c [\text{kg/m}^3]$	380
Aggregate/cement ratio	a/c	4.79
Water/cement ratio	w/c	0.60
Compressive strength	$f_{c,\infty}$ [MPa]	19.8
$f_c(\xi) = f_{c,\infty} \frac{\xi - \xi_0}{1 - \xi_0}$	ξ ₀	0.01
Biaxial compres, strength factor	f_b/f_c	1.16
Young's modulus	E_{∞} [MPa]	20400
$E(\xi) = E_{\infty}\sqrt{\xi}$		
Poisson's ratio	v	0.2
Characteristic time $\tau_w(\xi) = \xi \cdot \tau_{w\infty}$	$\tau_{w_{i}\infty}$ [h]	24
Viscous compliance	J^{v0}_{∞}	$127 \cdot 10^{-6}$
$J_{\infty}^{\nu} = J_{\infty}^{\nu,0}(1-\xi)$	[1/MPa]	
Softening modulus for flow creep	H[MPa]	$1/7 \cdot 10^{6}$
Chemical affinity	<i>a</i> _A [1/s]	7.313
$\widetilde{A}(\xi) = a_{A} \frac{1 - exp(-b_{A}\xi)}{1 + c_{A}\xi^{d_{A}}}$	b_A	10.46
	c_{A}	169.3

	$d_{ m A}$	4.37
Chemical dilation angle	a_s	$-0.405 \cdot 10^{-3}$
$\beta(\xi) = a_s + b_s \xi$	b_s	$0.943 \cdot 10^{-3}$

thermo-chemical analysis for determination of the temperature field and the field of degree of hydration and a subsequent mechanical analysis.

The mixture characteristics and the material properties of the considered shotcrete are listed in Table 1.

3 SOLUTION OF THE THERMO-CHEMICAL PROBLEM

During the hydration of shotcrete, the hydration heat l_{ξ} is released, resulting in an increase of the temperature in the shotcrete lining and, hence, in heat conduction into the surrounding ground and heat radiation towards the tunnel cavity. The field equation for the thermo-chemical problem is given by (Ulm & Coussy 1995)

$$\rho cT - l_{\xi} \xi = \operatorname{div} \mathbf{q}, \tag{9}$$

with ρ as the density and c as the heat capacity. q is the heat flow vector. It is related to the temperature via the linear law of Fourier:

 $\mathbf{q} = -k \cdot \operatorname{grad} T$

(10)

where *k* is the thermal conductivity.

3.1 Structural model, boundary conditions, and FE discretization

For determination of the temperature profiles in consequence of the hydration process, a 1D axisymmetric FE model is employed (Fig. 4). The respective FE formulation for the solution of the axisymmetric thermo-chemical problem can be found in (Lackner & Mang 2002). The FE model refers to a section perpendicular to the axis of the tunnel. It comprises both the shotcrete lining and the surrounding ground. The thickness of the shotcrete lining is set equal to 30 cm. It is discretized with 5 finite elements in the radial direction. The surrounding rock is discretized with 36 finite elements.

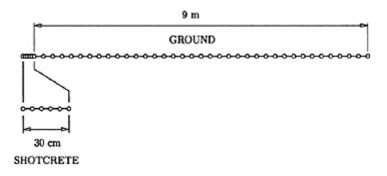


Figure 4. Thermo-chemical analysis: 1D axisymmetric FE mesh.

At the inner surface of the shotcrete lining, heat radiation from the lining to the tunnel opening is considered. The underlying heat radiation law reads:

 $q = \alpha_R(T - T_\infty)$

(11)

with T standing for the temperature at the inner surface of the lining and T_{∞} for the temperature of the air in the tunnel cavity. α_R is the radiation coefficient.

3.2 Material properties and initial temperature

The thermal properties of the shotcrete and the ground employed in the numerical analysis are listed in Table 2.

-	-	
Shotcrete		
Heat capacity	$\rho c [kJ/(m^3 K)]$	2428
Thermal conductivity	<i>k</i> [kJ/(m h K)]	12.6
Heat of hydration	$l_{\xi} [\mathrm{kJ/m}^3]$	190000
Radiation coefficient shotcrete-air	$\alpha_R [kJ/(m^2 h K)]$	40
Ground		
Heat capacity	$\rho c [kJ/(m^3 K)]$	2300
Thermal conductivity	<i>k</i> [kJ/(m h K)]	7.2

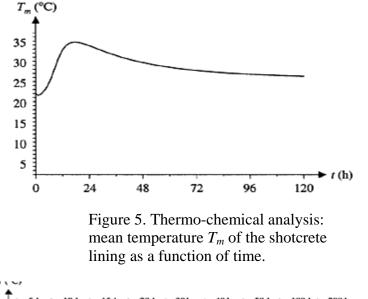
Table 2. Thermo-chemical analysis: material parameters for shotcrete and ground.

The initial temperatures of the shotcrete lining and the ground are set equal to 20 and 10°C, respectively. The temperature in the tunnel opening, T_{∞} , is set equal to 25°C.

3.3 Presentation of results

Figure 5 shows the evolution of the mean temperature T_m in the shotcrete lining as a function of time. The maximum value of $T_{m\nu}$ 32.5°C, is reached about 18 hours after installation of the lining.

The distribution of the temperature in the shotcrete lining and in the adjacent ground at different time instants is given in Figure 6. Temperature profiles like this serve as input for the following mechanical analysis.



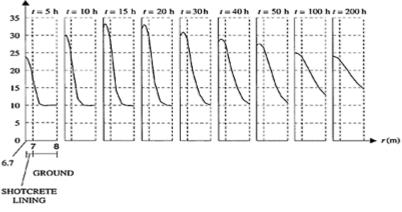


Figure 6. Thermo-chemical analysis: distribution of the temperature T in the shotcrete lining and in the ground at different time instants.

4 SOLUTION OF THE MECHANICAL PROBLEM: NUMERICAL MODEL

4.1 Structural model and FE discretization

The axisymmetric model employed for the solution of the mechanical problem allows to simulate both a continuous excavation of the tunnel with a driving speed of 2.5 m/d and the construction break during the summer holidays.

As pointed out in (Boldini et al. 2003), an excavation length of 10R considered in the structural model, where R is the radius of the tunnel, is sufficient to avoid boundary effects influencing the numerical solution. Accordingly, the length of the structural model was set equal to 20R (Fig. 7b). The excavation starts from the right boundary and is stopped when the tunnel face reaches the center of the model, i.e., after the excavation of 10R of tunnel length. Continuation of the analysis for three weeks, during which the excavation was interrupted, allows to simulate the situation during the summer holidays.

A local coordinate system is introduced. The coordinate r refers to the radial direction and the coordinate z to the longitudinal direction of the tunnel. z=0 refers to the final location of the tunnel face.

A circular tunnel with a radius R=7 m is chosen for all the numerical analyses (Fig. 7a). After the excavation, the application of a closed 30 cm-thick shotcrete shell is considered.

Figure 8 shows the FE discretization consisting of 15266 axisymmetric finite elements. Near the tunnel face, the mesh is refined in order to attain better approximations of the rather high stress and strain gradients in this area.

4.2 Excavation scheme

The excavation of the tunnel is simulated by replacing the respective ground elements by cavity elements. The latter are characterized by marginal stiffness. Application of shotcrete is modeled by replacing the respective cavity elements by finite elements with shotcrete characteristics. Similar to the FE discretization employed for the thermochemical analysis, the shotcrete shell is discretized in the thickness direction by means of five finite elements.

The length of one excavation step and, hence, the length of the unsupported part of the tunnel is set equal to 1 m.

The excavation rate is set equal to 2.5 m/day. The time assigned to complete 1 m of tunnel is divided into two parts: 2/3 of the time is dedicated to the excavation and the remaining 1/3 to the application of shotcrete. For an excavation rate of 2.5 m/day, 1 m of the tunnel is completed in 9.6 hours. Consequently, 2/3.9.6=6.4 hours are assigned to the excavation process and 1/3.9.6=3.2 hours to the application of the shotcrete lining.

After the tunnel face has reached its final position at z=0 m, the excavation is stopped. The analysis, however, is continued for three weeks in order to simulate a typical break period during summer holidays.

4.3 Initial state of stress

Consideration of axisymmetric conditions implies that the initial state of stress is isotropic. The isotropic stress is set equal to $\sigma_0=2.8$ MPa. This value is appropriate, e.g., for representing the *in situ* stress conditions of a tunnel at a depth h=150 m characterized by a ground weight per unit volume $\gamma=25$ kN/m³ and a lateral pressure coefficient $K_0=0.5$ (Boldini et al. 2003).

In the structural model, the initial stress state is introduced by setting the principal stresses to 2.8 MPa in all finite elements and by applying a constant pressure p_0 at the top boundary of the model, with $p_0=2.8$ MPa (see Fig. 7b).

4.4 Consideration of dowels within the FE model

In the FE model, fiber-glass dowels with a length of 24 m are considered. They are discretized by means of

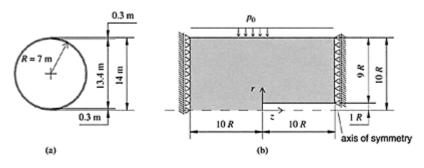


Figure 7. Mechanical analysis: (a) cross section of the tunnel and (b) geometric dimensions of the structural model.

chains consisting of two-node truss elements. The nodes of each truss element are connected to the respective nodes of the ground elements. Accordingly, no slip between the dowels and the ground is considered which, for the case of cemented fiber-glass dowels under moderate loading, is a reasonable assumption.

The distance between two subsequent settings of dowels is set equal to 13 m. For an excavation length of 70 m, as considered in the structural model (Fig. 7b), the setting of dowels is simulated at the following positions of the tunnel face: z=65, 52, 39, 26, 13, and 0 m.

The layout of the face reinforcement considered in the numerical analysis is illustrated in Figure 9 for installation of the dowels at z=0. The location of two consecutive face reinforcements differs by the radial location of the dowels. The radial distance between two corresponding sets of dowels is 35 cm.

In an axisymmetric analysis, the dowels, which are set at different locations in the tunnel face, must be shifted towards the axisymmetric plane (Fig. 10). Consequently, at

each location of a dowel indicated in Figure 9, several dowels characterized by the same distance from the axis of symmetry are considered. A total number of 45 dowels is placed during a single reinforcement step in all the numerical analyses. Accordingly, each set of the different dowels of the

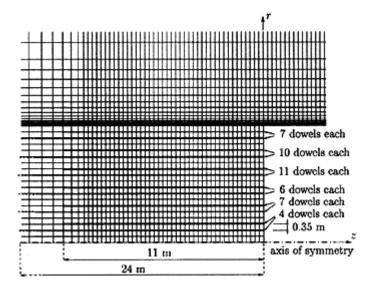
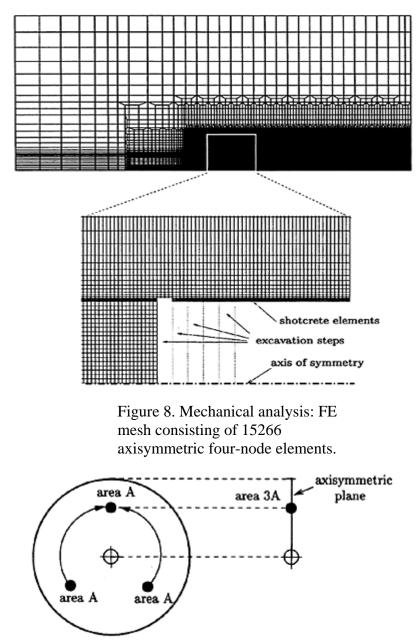


Figure 9. Mechanical analysis: location of truss elements representing fiberglass dowels of 24 m length used for face reinforcement.



view onto tunnel face

Figure 10. Mechanical analysis: an example of shifting three dowels towards the axisymmetric plane.

Young's modulus	E [MPa]	15000
Poisson's ratio	v[-]	0.3
Yield strength	f_y [MPa]	900

Table 3. Mechanical analysis: mechanical properties of the fiber-glass dowels.

axisymmetric model represents 4, 7, 6, 11, 10, and 7 real dowels (Fig. 9).

The time assigned to the installation of the dowels at the tunnel face is fixed as 3 days. For the simulation of the mechanical behavior of the dowels, a linear elastic—ideally plastic material model is employed. The mechanical properties of the fiber-glass dowels used in the numerical analysis are given in Table 3.

5 SOLUTION OF THE MECHANICAL PROBLEM: PRESENTATION OF RESULTS

5.1 Performed analyses

The focus of the analyses performed in this study is on the investigation of the behavior of tunnels excavated in squeezing ground conditions. For this purpose, the ground properties were chosen in such a way as to match conditions indicating a high squeezing potential, with $f_c/\sigma_0 < 0.2$. Four different types of ground were considered, characterized by the following values for the ground cohesion: 10 (ground type A), 50 (ground type B), 100 (ground type C), and 150 kPa (ground type D). The remaining material parameters were the same for all ground types. Their values are reported in Table 4.

In order to assess the effect of the shotcrete lining and fiber-glass dowels on the static response of the tunnel, four different analyses were performed for each ground type condition:

1. In the first analysis, neither the shotcrete lining nor the fiber-glass dowels are considered. In this analysis the behavior of the unlined tunnel is investigated;

1 1 8		
Friction angle	φ[-]	18°
Dilation angle	$\psi[-]$	1.8°
Young's modulus	E [MPa]	500
Poisson's ratio	v [-]	0.3
Relaxation time	au [h]	0.5

Table 4. Mechanical analysis: mechanical properties of the ground.

Cohesion c [kPa]	10	50	100	150
Squeezing potential f_c/σ_0	0.01	0.05	0.10	0.15
Unlined tunnel	A1	B1	C1	D1
Shotcrete only	A2	B2	C2	D2
Shotcrete+0.29 dowels/m ²	A3	B3	C3	D3
Shotcrete+0.58 dowels/m ²	A4	B4	C4	D4

Table 5. Mechanical analysis: synopsis of the performed analyses.

2. The installation of shotcrete as primary lining is considered in the second analysis;

3. Both the shotcrete lining and the reinforcement of the tunnel face by means of fiberglass dowels are considered in the third analysis. The reinforcement density is set equal to 0.29 dowels m² (tunnel face);

4. Finally, the fiber-glass dowel density is increased to 0.58 dowels/m² (tunnel face).

The synopsis of the full set of performed analyses is given in Table 5.

5.2 Influence of the support

In this subsection, the influence of the different support means (primary lining of shotcrete, face reinforcements by fiber-glass dowels) is investigated for ground type B (cohesion c=50 kPa).

Figure 11 shows the longitudinal displacement u_z/R of the ground ahead the tunnel face (z<0) at r=0, obtained at time instants (a) t=1032 h and (b) t=1608 h. At t=1032 h, the tunnel face has reached its final position at z=0 for a driving speed of 2.5 m/days and five stops of excavation of three days for the installation of the face reinforcement ($t=70/2.5+5\cdot3=43$ days=1032 hours). In order to permit a comparison between the different analyses, the same excavation stops are considered in the analyses B1 and B2, even though no setting of fiber- glass dowels is taken into account. Between the time instants t=1032 h and t=1608 h, the position of the tunnel face remains unchanged in the numerical analysis, allowing simulation of a three-week construction break in consequence of summer holidays or an unexpected stop of advancement.

The analysis B1 gives the highest value for u_z/R , with $u_z/R=10\%$ at the face after the three-week break,

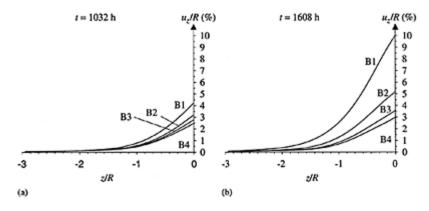


Figure 11. Mechanical analysis: longitudinal displacement u_z/R (%) at r=0 obtained at time instants (a) t=1032 h and (b) t=1608 h for the analyses B1, B2, B3, and B4.

indicating the likely collapse of the tunnel. The influence of the primary lining (analysis B2) is remarkable (even ahead of the tunnel face), reducing u_z/R to almost one half for both steady-state excavation (Fig. 11a) and temporary stops (Fig. 11b). The influence of face reinforcement is not significant at the time instant t=1032 h. On the contrary, the longitudinal displacements of the ground are considerably reduced by the fiber-glass dowels after the stop of the excavation, resulting in the stability of the tunnel face. Doubling the dowel density (analysis B4) results in a reduction of u_z/R of 0.5% as compared to analysis B3.

The loading of the reinforcement system is illustrated in Figure 12 for the analysis B3 in terms of the average stress in the fiber glass dowels σ_{dm} :

$$\sigma_{dm} = \frac{1}{n_d} \sum_{i=1}^{n_d} \sigma_{d,i}$$
(12)

where $\sigma_{d,i}$ is the stress in the i-th dowel, and n_d represents the number of dowels at the considered cross section of the tunnel. In the area of overlapping dowels (see Fig. 9), n_d =90. Otherwise, n_d =45. Figure 12 shows the distribution of σ_{dm} during the tunnel excavation from z=13 m to z=0 m. A new set of fiberglass dowels is installed at z=13 m. The the previous one (installed at z=26 m) extends to z=2 m. Hence, between z=13 m and z=2 m, n_d =90. In the course of the excavation, the length of the dowels installed at z=26 m and z=13 m continuously decreases, resulting in an increase of the average stress. At z=2 m, one set of dowels ends, explaining the jumps in the distribution of σ_{dm} . For all excavation steps referred to in Figure 12, significant loading of the dowels is observed only within a distance ahead of the face that is equal to the tunnel radius *R*.

Figure 13 shows the stress σ_d in the set of fiber-glass dowels located at r=1.05 m from the tunnel axis (see

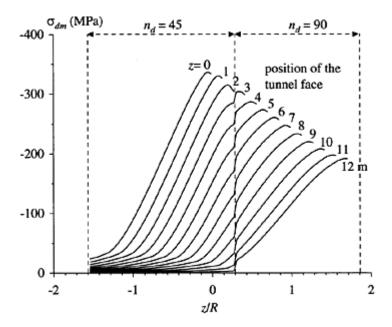


Figure 12. Mechanical analysis: average stress σ_{dm} in the fiber-glass dowels during the excavation from z=13 m to z=0 m for the analysis B3 for different position of the tunnel face z=12, 11, ..., 0 m (n_d =number of dowels).

Fig. 9), obtained at time instants t=1032 h and t=1608 h. Even though the face is reinforced with a new set of fiber-glass dowels before the excavation break of three weeks, resulting in a total number of 90 dowels installed ahead of the tunnel face, the continuation of stress release in the ground results in a significant increase of σ_d .

Figure 14 shows the radial displacement u_r/R of the tunnel wall (r=R) as a function of the distance from the tunnel face z/R, obtained at time instants (a) t=1032 h and (b) t=1608 h. For the analysis considering the application of shotcrete, a saw-toothed shape of u_r/R is observed. Each saw-tooth refers to one excavation step of 1 m. The change of u_r within one saw-tooth indicates the variation of deformation and loading within 1 m of the tunnel.

The installation of the shotcrete lining (analysis B2) led to a large reduction of the radial displacements as compared to the result obtained for the unlined tunnel (analysis B1). For the latter, the collapse of the tunnel is likely to occur within a short distance from the tunnel face. Almost identical distributions for the radial displacements u_r/R are obtained for the analyses B3 and B4 for z/R>0, at both time instants t=1032 h and t=1608 h, indicating a negligible influence of the face reinforcement on the convergence of the

tunnel. Some differences are visible for z/R<0, where the dowels result in a reduction of the radial displacements proportional to

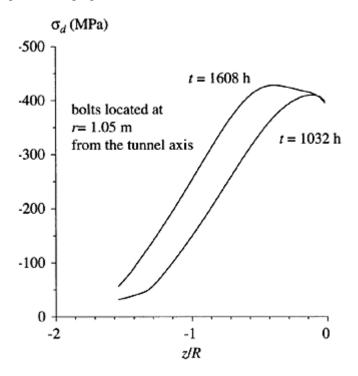


Figure 13. Mechanical analysis: stress σ_d in the fiber-glass dowels located at r=1.05 m from the tunnel axis obtained at time instants t=1032 h and t=1608 h.

the reinforcement density. The effect of the excavation break is concentrated near the tunnel face (Fig. 14b), where the relatively high compliance of the young shotcrete, for z/R>0, results in an increase of u_r/R .

For the analyses B2, B3, and B4, a continuous, slight increase of the radial displacements is observed. As pointed out in Boldini et al. (2003), this increase is a consequence of chemical shrinkage of shotcrete. At the locations of the tunnel corresponding to the installation of the face reinforcement, a localized increase of the radial displacements u_r/R was obtained in all the analyses. This increase is explained by the creep of the ground during the threedays break considered in the analyses to simulate the installation of the face reinforcement.

The distribution of the hoop force n_{ϕ} in the shotcrete lining (normalized by the maximum compressive axial force in the shotcrete lining $n_{\infty}=f_{c,\infty}\cdot h=19.8\cdot 0.3=5.94$ MN/m) is depicted in Figure 15 for time instants (a) t=1032 h and (b) t=1608 h. The average hoop force in the lining is very close to the maximum compressive axial force n_{∞}

of the lining, resulting from the severe squeezing conditions simulated in the numerical analyses. Creep of shotcrete results in a slight reduction of the compressive loading in the course of the three-day excavation break for the installation of the face reinforcement. The increase of stiffness during the three-day break leads to a sharp increase of loading after continuation of the excavation. There are no significant differences between the results obtained in the analyses B2, B3, and B4 at time instant t=1032 h even if face reinforcement leads to a small increase of the compressive force in the shotcrete lining in analyses B3 and B4 (Fig. 15a). After the excavation break of three weeks, the plotted

distributions are almost unchanged for z/R>1 whereas a significant increase of n_{ϕ}/n_{∞} is

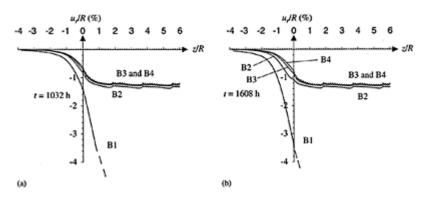


Figure 14. Mechanical analysis: radial displacement u_r/R (%) at r=R obtained at time instants (a) t=1032 h and (b) t=1608 h for the analyses B1, B2, B3, and B4.

observed near the tunnel face. The increase of hoop force near the tunnel face is influenced considerably by the density of the face reinforcement: the smaller the dowel density, the larger the relaxation of the ground during the excavation break and the greater the load on the shotcrete lining (Fig. 15b).

Figure 16 shows the distribution of the longitudinal force n_z in the shotcrete lining (normalized by the maximum compressive axial force n_∞), for time instants (a) t=1032 h and (b) t=1608 h. According to Boldini et al. 2003, there are four reasons for the loading of the lining in there longitudinal direction: close to the tunnel face, (a) excavation-induced bending of the ground-shotcrete compound structure results in compressive loading which is further increased by (b) compressive hoop force via Poisson's effect; the compressive loading is reduced by (c) chemical shrinkage and (d) deformations in the longitudinal direction in consequence of the excavation. Analysis B4 results in a greater value for the compressive longitudinal force. This is associated mainly with the more pronounced excavation-induced bending of the shotcrete lining and the higher compressive hoop force via Poisson's effect. The stress release in the ground at the tunnel face during the three-week break results in a strong increase of the compressive

longitudinal force in the final shotcrete segment installed at $0 \le z \le 1$ m, which is more significant for analysis B2, where no face reinforcement is considered (see Fig. 16b). For z/R < 2, chemical shrinkage of shotcrete results in a reduction of the compressive loading in the longitudinal direction.

5.3 Influence of the ground cohesion

The influence of the ground cohesion (i.e., of the ratio f_c/σ_0) and of the density of the face reinforcement on the static response of the tunnel is shown in Figure 17.

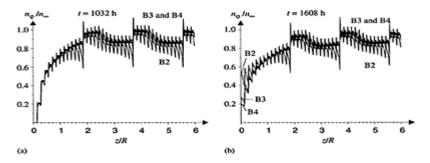


Figure 15. Mechanical analysis: distribution of compressive hoop force n_{ϕ}/n_{∞} in the shotcrete lining obtained at time instants (a) *t*=1032 h and (b) *t*=1608 h for the analyses B2, B3, and B4 ($n_{\infty}=f_{c,\infty}\cdot h=19.8\cdot 0.3=5.94$ MN/m).

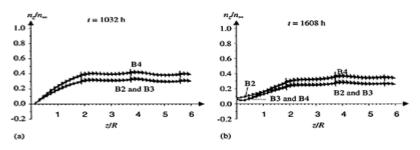


Figure 16. Mechanical analysis: distribution of compressive longitudinal force n_z/n_∞ in the shotcrete lining obtained at time instants (a) t=1032 h and (b)t=1608 h for the

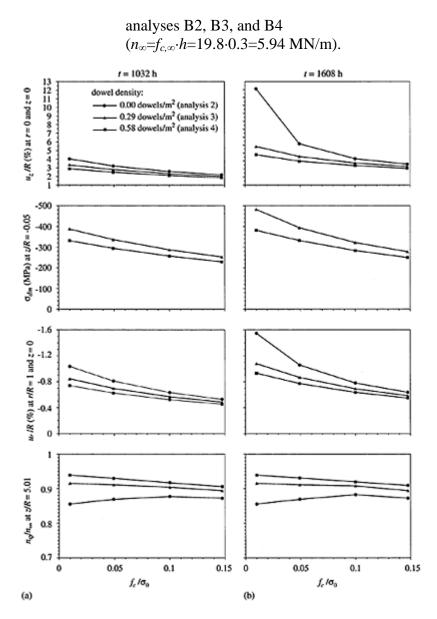


Figure 17. Synopsis of the numerical results accounting for the influence of ground cohesion and the density of face reinforcement.

Column (a) refers to the numerical results obtained at time instant t=1032 h while column (b) reports the results obtained at t=1608 h, after having stopped the face advance for three weeks.

Face reinforcement turns out to be very effective in reducing the longitudinal displacement u_z/R at the tunnel face (r=0 and z=0) for very strong squeezing conditions ($f_c/\sigma_0=0.01$ and 0.05), especially when relaxation of the ground is allowed during the excavation stop. In this sense, the ground viscosity is found to improve the face stability during tunneling under squeezing conditions. The reduction of the face deformations by face reinforcement becomes less pronounced for increasing values of f_c/σ_0 .

The more bolts are installed during face reinforcement, the smaller the average stress in the dowels (the maximum value obtained at z/R=-0.05 is reported in Fig. 17). The average stress in the dowels decreases almost linearly as the ground cohesion increases.

The radial displacement ur/R of the tunnel wall at the face (z=0 and r/R=1) reflects the same influence of the ground cohesion and the reinforcement density as already observed for the longitudinal displacements. Radial displacements are significantly smaller than longitudinal displacements, thanks to the immediate support of the shotcrete lining applied up to the face every meter of excavation.

Finally, the normalized hoop forces in the shotcrete lining at a great distance from the face (z/R=5.01) are illustrated in Figure 17. The increase in final load acting on the lining for higher dowel density stems from the reduced stress relaxation of the ground surrounding the face in consequence of face reinforcement. No substantial modifications are induced in the shotcrete lining during the three-week stop of tunnel excavation.

6 SUMMARY AND CONCLUSIONS

In this paper, tunneling based on full-face excavation, reinforcement by fiber-glass dowels and immediate support with a primary lining of shotcrete was investigated numerically. Different values were taken for the ground cohesion in order to account for different squeezing potentials during tunnel excavation.

The numerical analysis was performed in two separated steps. First, the thermochemical problem related to the hydration process of shotcrete was solved. Then, the mechanical problem was treated. It allowed to estimate the state of stress and deformation in the ground, the shotcrete lining, and the fiber-glass dowels.

The primary lining of shotcrete was found to play an essential role, assuring the stability of the tunnel in squeezing ground condition by reducing both the longitudinal and the radial displacements in the ground.

The effectiveness of fiber-glass dowels concerning reduction of face deformations of the tunnel was remarkable in case of poor ground conditions. It was essential during the excavation stop, avoiding the collapse of the tunnel face by the creeping ground.

The reduction of ground deformations during tunneling by face reinforcement results in an increase of the final load in the shotcrete lining.

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Risk-based design using numerical modelling

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ABSTRACT: Traditionally, tunnels have been designed using empirical methods and a semi-observational approach. A series of high-profile collapses in civil tunnelling however, have demonstrated that these methods cannot be used in isolation and that increasingly, designers need to take a risk-based design approach and to make more use of sophisticated numerical analysis. There is now general acceptance in the UK that risk management is not optional. It took some time for this realisation to sink in, but the Construction and Design Management Regulations (CDM) implemented in 1994 made it mandatory and enforceable. Therefore it has been necessary to incorporate strategies into the design and construction phases of a project that provide both continuity and a framework for an open process that can be adjusted during the progress of the works to suit the actual conditions. This paper looks at some of the concepts and definitions that are helpful in this process.

Other developments such as The Joint Code of Practice for Risk Management of Tunnel Works, are discussed. This aims to promote and secure best practice for the minimisation and management of risks associated with the design and construction of tunnels and other underground structures. While the paper will be illustrated with examples from civil engineering projects where increasingly Risk Management is becoming a pre-requisite to major projects, the authors believe that the general principles have a broad applicability to underground works, including mining.

1 INTRODUCTION

This paper will attempt to explain how the risk-based design approach has been introduced to tunnelling in the UK in recent years. The current application will be presented and its benefits will be illustrated through examples from real projects. Numerical modeling has a valuable role to play in risk-based designs and this will be

explained in the paper. This design approach is powerful and can be applied in many situations beyond tunnelling. Crucially it looks at how modeling needs to be supported with sound engineering judgment and the identification and assessment of risk. These approaches are complementary and cannot exist in isolation.

2 WHY A RISK-BASED DESIGN APPROACH?

There is an axiom that 'if it ain't broken, then don't fix it'. So is there really a need for a new design approach?

2.1 Current design practice

Traditionally tunnel design has relied heavily on empirical design methods and the experience of the tunnel engineers. The disadvantage of the empirical approach is that engineering judgment and empirical design guides may be inadequate when applied in new situations, in situations beyond the experience of the engineers or the basis of the design guide.

As analytical design methods have improved, particularly computer-aided calculations, there has been a shift towards a deterministic approach to tunnel design. In other words, calculations are done and on the basis of them a design is specified. A blind reliance on the results of calculations, at the expense of engineering judgement, exposes a weakness in this approach. All analytical models are simplifications of reality and therefore the results of calculations are only estimates of how a tunnel will behave.

2.2 The costs of failure

There have been many technological innovations in tunnelling in recent years, particularly with regard to soft ground tunnelling and the use of Tunnel Boring Machines (TBM's), where applying face pressures has been extremely efficient in controlling face losses and hence surface settlements, particularly in high risk urban areas. Such construction methods have moved the industry a long way towards achieving efficient and safe tunnelling. However, it also has to be recognized that the magnitude of cost overruns when problems arise during the progress of tunnelling work can be out of all proportion to the original cost of the construction. One recent example of a TBM driven tunnel in the UK led to a 4200% increase in cost/m to remediate a section of failed tunnel supported using a pre-cast concrete segmental lining. While this is exceptional it is a fact that the cost of replacing new build which fails is at least a factor of 2, and often much more, when compared to the original.

As a result of incidents like this, those at Heathrow Express² in 1994 and, latterly Dulles Airport in the USA, the insurance industry has been looking hard at their risk and liability on major tunnelling projects.

Current statistics indicate that the leading tunnel insurer's ratio of premiums received to claims and payouts has resulted in losses of between 500–1000% in the last 5 years. The outcome of this was inevitable with many insurers refusing to accept new tunnelling

business in the belief that many contractors were simply using this to allow them to accommodate a higher level of risk in their bids and methods of working.

2.3 Statu tory framework

In addition in the UK the introduction of the Construction and Design Management Regulations (1994) has compelled clients, designers and constructors both to consider risks explicitly and to communicate those risks to all the parties involved.

One motivation for this change was the realisation that many accidents could be avoided if the hazard had been removed at the design stage.

The regulations envisage a hierarchy of protective measures. Ideally the hazard should be eliminated completely. Failing that the mitigation measures should protect all who are exposed to the hazard. Finally the lowest level of mitigation is protection of the individual.

For example, considering the case of sprayed concrete, alkaline accelerators should be avoided completely to remove the hazard, rather than specifying additional personal protective equipment.

2.4 UK Industry Joint Code of Practice

In view of the losses, it came as no surprise that contractors were forced into negotiations with the insurance industry, leading to the recent publication of The Joint Code of Practice for Risk Management of Tunnel Works² in the UK produced jointly by the British Tunnelling Society (BTS) and The Association of British Insurers (ABI). This aims to promote and secure best practice for the minimisation and management of risks associated with the design and construction of tunnels and other underground structures. It aims to improve the management of risk on projects in order to try and control losses. It also clearly seeks to limit an insurers' liability to no more than the equivalent of the original construction cost. This is a tight and extremely onerous limitation but should be seen in the general context of insurers realising that many contractors are using them as a shield. This is borne out by statistics which show that for some years claims and payouts have substantially exceeded income. The response of the insurance industry is hardly surprising but the implications are far reaching. They will be felt worldwide and will affect the way in which clients procure projects, the risks that they will accept and the means by which they will expect their designers and contractors to manage those risks on their behalf.

The questions that arise from this development are therefore:

- What constitutes risk management on major projects
- What impact will it have on clients, including procurement of contracts and acceptance of risk.

2.5 Impact of collapse investigations

Investigations into recent collapses have shown that the sources of risk (hazards) have commonly been overlooked and that risks have frequently not been controlled through 'defensive' systems (i.e. preventive management systems). It was found that the failure of TBM tunnels at Portsmouth and Hull³, in the UK, featured common risk factors. Specific recommendations for closed face TBM operations were that careful consideration needs to be given to a specific ground types, TBM operation and design.

The collapse of NATM tunnels at Heathrow airport in 1994 was classified as an 'organisational accident', where errors were made leading to poor design and planning, a lack of quality during construction, a lack of engineering control and significantly, a lack of safety management.

Other collective lessons learned have been:

- Potential for major accidents must be recognised through use of hazard identification, consequence analysis and risk reduction strategies;
- New technologies need to be thoroughly understood and tested before being implemented;
- Production pressures need to be balanced by defensive precautionary systems.

3 WHAT IS A RISK-BASED DESIGN?

The elements of risk-based design are illustrated diagrammatically in Figure 1. This shows how Project

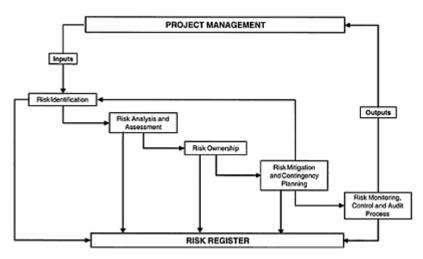


Figure 1. Risk-based design process.

Management provides inputs to a sequence of key activities whose outputs populate a risk register which is key to ongoing project management.

3.1 Risk management

Risk Management is the overall application of policies, processes and practices dealing with risk. In reality it means providing an auditable approach to assessing, analysing and managing risk during design and construction to ensure that the works are carried out safely and in accordance with a reasonable programme and budget. If projects are unable to meet a construction programme this could arise from a number of causes some of which are not related to hazards in the conventional sense, e.g. a tunnel failure due to unforeseen local conditions, but more to inexperience on the part of a contractor or designs which may be difficult to construct. However, this only serves to emphasize that in procuring a major contract clients need to be aware that safety and preventing major collapses is only part of a much wider process that should be all embracing as far as the project is concerned. The results of a recent survey in the UK showed that ground-related risks are substantial, with groundwater problems accounting for 13% of them, soil properties 20% and ground geometry 22%. These risks may impact on cost, health and safety, the environment, programme and quality.

3.2 Definitions

Hazards: The source of risk is a thing or activity with a potential for consequence. Typically the focus is on threats such as unforeseen geological conditions and it is often difficult to investigate and forecast problems for long tunnel drives in remote areas. However, increasingly it is necessary to evaluate and provide a means for coping with problems that might arise. It is often convenient for designers and contractors to consider most problems under the guise of 'unforeseen conditions', but more often than not it is simply a lack of recognition at the advancing face of the response of the rockmass to excavation. In the future this type of claim will be more difficult to reach agreement on with the insurers.

However, there is also a positive side. Hazards can be overcome successfully and turned into opportunities and very often experienced contractors will work closely with the designers to achieve real benefits that can be passed on to the client. This generally requires a partnering type approach rather than the more traditional forms of contract that can lead to confrontation, particularly on projects where the client and designers seek to pass all the risk onto a contractor.

What it is important to recognise is—'if a hazard is not identified then it leads to events that cannot be controlled'—and this is when problems with programmes and budgets, and possibly safety issues, escalate and become difficult to manage on a project. **Risk:** Risk is an adverse event having a probability of occurrence and an impact that will affect the achievement of the project's objectives. It is the combination of the chance of an event occurring and its consequences. There will be a level of risk associated with each hazard. The level of acceptable risk will depend on both the probability of an event occurring and the severity of the impact on the project and is obtained using a formal risk assessment. It is the most important tool that the client and designers have at their disposal. This is formalisation of a process that in the past was built into a design or approach to construction using judgment and experience. The risk evaluation can either be quantitative (e.g. Monte Carlo analysis) or semi-quantitative and the level of risk will determine the extent to which measures are taken to mitigate the impact of a hazard on a project. In this context it is also important to note that where risks are 'As Low As Reasonably Practical', i.e. the ALARP principle, and the cost of controlling or eliminating the risk is out of all proportion to the cost of the project, they should be regarded as residual and both acceptable and manageable. A range of tools are available to assist in the identification of risks, and use may be made of network analyses, workshops or brainstorming. Higher-level legal, economic and financial risks are generally the first to be considered, followed by construction, safety and programme risks.

3.3 Risk identification

The first step in Figure 1—risk identification—is perhaps the most important one. If a risk is not identified then the project will not take any steps to control it. It is important to consider the risks during operation and maintenance of the tunnel as well as just construction.

Numerical modelling can then be used to explore the severity of the risks and to examine which mitigation measures are most effective. It must be remembered that numerical models are only estimates of how a tunnel might behave because of the simplifications that one must make when constructing the model. For example, the three-dimensional nature of the tunnel heading is often simplified and modelled in a two-dimensional analysis. The modelling of the ground and the construction sequence can have a large influence on predicted loads in the lining (Thomas et al, 2004) as shown in Figure 2. Therefore, while sophisticated numerical modelling can reduce the effect of the simplifications, sound engineering judgement is always required when interpreting the results from numerical modelling.

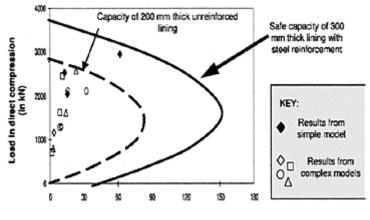
Numerical modelling can also be combined with stochastic methods (Thomas et al, 2004). Basically stochastic methods involve repeatedly performing the same calculations with key input parameters varying randomly within their likely ranges. The results for key output parameters—e.g.: factor of safety on stability or maximum surface settlement—can be gathered together from all the results and expressed as a probability distribution. The designers can then see how likely it is for an outcome to occur—e.g.: there is a 5% chance that settlement will exceed value X. Stochastic methods attempt to create a more realistic modelling of the real case, in which the exact values of parameters (especially geotechnical parameters) are not known and at best the designer knows that the values are likely to lie within a certain range.

3.4 Tools

Risk Assessment: This is a formal process that needs to be conducted at the start of the design phase of a project and generally involves a brainstorming session with experienced staff. Involving the client and obtaining his inputs are extremely important at this stage. It is generally best managed using a facilitator but this is not essential if there is an experienced project manager to drive the process. Ranking of risks in terms of likelihood and impact (application of risk criteria) will enable semi-quantitative Risk Exposure values to be tabled.

The aim of the process therefore is not only to identify the hazards but to demonstrate that in the design phase, these are recognised and the measures taken by the designers have reduced the level of risk to manageable proportions. More importantly residual risks need to be flagged up with the contractor and considered in his excavation and support methods through a 'Live Risk Register'. Implicit in this process is the need to identify Risk owners as well as the point in the project when the risk is likely to occur.

Risk Register: As a live document it is available to all parties involved in construction, built with key inputs from the project management team and providing key outputs to it. It is based on the identification of risks



Load in bending (in kNm)

Figure 2. The effect of the constitutive model of the sprayed concrete lining on predicted loads (Thomas et al, 2004).

and their ranking in terms of likelihood and impact. This ranking is possible by applying an accepted risk criteria to determine Risk Exposure values and is a quick and semiquantitative approach that may be refined by means of simulations, using the Monte Carlo approach. Further inputs to the risk register are assigning risk ownership and specifying when in the project the particular risk is likely to occur. This whole process allows risk mitigation and contingency planning strategies to be developed, which in themselves may generate further risks which are entered into the risk register and reanalysed. The Register should be part of the contractors Site Safety Plan as well as an integral part of his Quality Control/Assurance System. This is what the insurance industry now expects.

3.5 Integration of design & construction

As noted earlier, a prime motivation for the UK's CDM safety regulations was to improve communication between the designers and the construction team. It is inevitable that the designers will be unable to remove all the risks from the design of a tunnel. Therefore it is important that the 'residual' risks are communicated to the team that is building the tunnel. The construction team can then take steps to remove or mitigate those residual risks.

A live risk register is one good communication tool but it may not be enough on its own. For example, in a tunnel constructed using a sequential excavation method with a sprayed concrete lining, there may be considerable scope for the construction team to vary the construction method. Support measures can be optimised to suit the prevailing ground conditions. However, it is important that the construction team understands the implications of changes in support or construction sequence.

4 CASE HISTORY—SDSU TUNNEL

A good example of a risk-based design that was successfully constructed is the sprayed concrete lined SDSU tunnel in San Diego, California. This short tunnel of only 330 m passes underneath the San Diego State University campus and carries a twin track light rail system (Thomas et al. 2003).

The 11 m span tunnel lies entirely within the Stadium Conglomerate, which is a dense deposit of cobbles in a sandy matrix. Very few tunnels have been built in this area and the behaviour of the conglomerate is not well understood.

4.1 Identifying risks

An initial assessment of the risks identified four areas of concern: the presence of water in the conglomerate which could lead to softening of the material; deep, loose sand lenses; the low cover at the portals and the potential for seismic activity in the area. It was also noted that this construction method—using sprayed concrete—was new to the region.

Different construction sequences at each of the key sections were examined using a numerical model and based on the results a range of support classes was specified. To cope with the complicated behaviour of the conglomerate, a sophisticated nonlinear model was used in the nurmerical models (Pound et al, 2003)⁶. This was calibrated against the results from in situ plate-bearing tests.

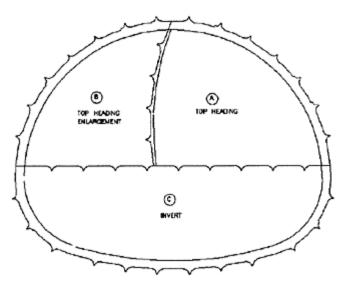
4.2 Risk mitigation measures

From the numerical modelling it was found that by subdividing the face into a sidegallery and enlargement (see Figure 3) the tunnel remained stable, even where water was present. Water around the tunnel was to be drained into it to reduce the water pressures.

The design included a range of additional support measures that could be applied if sand lenses were encountered.

At the low cover sections there was sometimes as little as 7 m above the 11 m span tunnel. Again subdivision of the face proved to be sufficient to maintain stability. Early completion of the whole lining (to close the structural ring) is important to minimise settlement in soft ground but in this case the conglomerate was strong enough for the Top Heading to be driven the entire length of the tunnel with construction of the Invert carried out after it was complete. This sequence is shown in Figure 3.

The seismic design was based on numerical models of the dynamic behaviour, checked against simple analytical guidelines for the relative stiffness of the structure compared to the ground.



EXCAVATION SEQUENCE 1

Figure 3. Cross-section for SDSU Tunnel

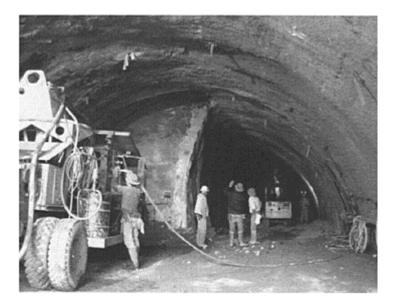


Figure 4. Side-gallery & enlargement, SDSU tunnel.

4.3 Risk management during construction

A representative of the designer was present throughout the construction period to assist the construction team and to evaluate proposed changes in support measures. The results from instrumentation in the tunnel and the ground were evaluated in a Daily Review Meeting and the support measures for the next day agreed. Within the support classes specified in the design the support could be varied to suit the prevailing ground conditions. The trends in monitoring data are as important as absolute values. This was a rigorously managed process to ensure that the safety of the works was always under control.

An additional benefit of the presence of the designer on site was that it promoted dialogue between the design and construction teams. Problems were identified early and solutions developed jointly.

4.4 Summary

As a result of the teamwork on site the tunnel was successfully completed according to programme in December 2002. The ground proved to be as stiff as the plate-bearing tests had suggested and there was very little water (see Figure 4). Very small settlements were recorded, in line with the original design predictions. Optimisation of the support and sequence enabled costs to be minimised and the effects of other delays on the project to be alleviated.

5 CONCLUSIONS

In many ways a risk-based design approach is the same as what has been regarded as 'good' design practice. The approach outlined above simply formalises the thought processes that good engineers have applied for years. The strength of this formalised approach is that it is rigorous and transparent. In other words, it is less likely that something will be overlooked and it is clear why design decisions have been made.

Numerical modelling has an important role to play in the assessment of geotechnical risks and the impact of mitigation measures on those risks. The numerical model should reflect the real case as far as is possible and practical. Stochastic methods offer the possibility to examine a situation more broadly and to estimate the probability of outcomes (e.g.: tunnel instability).

Whilst the need to apply rigorous risk-based design has evolved in UK civil tunnelling projects out of a series of high-profile collapses, the techniques and benefits of adopting such an approach are farreaching. There can be no civil or mining project that cannot benefit from an appraisal of economic, environmental or financial risks, nor construction, safety and programme ones. It remains to be seen to what extent this 'compulsory' approach becomes enforceable in other areas. With a realisation of the benefits however, it is suggested that risk management will yield rewards to the mining industry.

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Application of nondestmctive stress measurement technique for safety assessment of underground structures

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ABSTRACT: A magnetic anisotropy sensor is used for nondestructive measurement of stress on surfaces of a ferromagnetic material, such as steel. The sensor is built on the principle of the magneto-strictive effect in which changes in magnetic permeability due to deformation of a ferromagnetic material is measured in a nondestructive manner, which then can be translated into the absolute values of stresses existing on the surface of the material. This technique was applied to measure stresses of H-beams, used as tunnel support structures, to confirm expected measurement accuracy with reading error of about 10 to 20 MPa, which was confirmed by monitoring strains released during cutting tests. The results show that this method could be one of the promising technologies for non-destructive stress measurement for safe construction and maintenance of underground rock structures encountered in civil and mining engineering.

1 INTRODUCTION

A magnetic anisotropy sensor is used for nondestructive measurement of stress on surfaces of H-beams used as structural elements in tunnel support systems. The sensor is built on the principle of the magnetostrictive effect in which changes in magnetic permeability due to deformation of a ferromagnetic material, such as steel, are measured and converted to absolute values of stresses existing on the surface of the material. Proper treatment of boundary conditions allows determination of stress tensor completely on surface of H-beam flanges, for example. The level of estimation error, as investigated so far, is around 10 to 20 MPa, which is within an acceptable range considering that yielding and ultimate load capacity are discussed in the range greater than 250 MPa for normal steel.

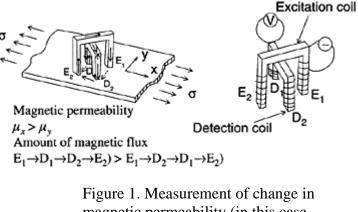
The sensor was used first to measure stresses on surfaces of four straight H-beams having the length of 2 m. Three of them were then bent in a factory to make curved H-beams for tunnel support with approximate radius of 6 m, for which the second

measurement was conducted. All four H-beams were then sent to a laboratory in which stress and strain measurements were conducted during loading experiments. One of the curved beams was then cut into five pieces during which released strains were monitored. Interpretation of the results from a series of stress measurements shows that (1) significant level of stresses exist already in straight H-beam, (2) alteration of stress level during a bending procedure is significant and part of H-beam after bending experience some plastic deformation, (3) residual stresses of significant order exist in H-beams even before they are placed in tunnels, and (4) changes of stress in H-beams during loading can be measured by the sensor with expected accuracy. These findings were confirmed by monitoring strains released during cutting tests.

Another series of stress measurement was conducted for H-beams in NATM tunnels. The results obtained from the field stress measurement were in full accordance with the findings made from the previous series of measurement, and showed the potential of this technology as promising tool for safety assessment of underground structures.

2 MEASUREMENT PRINCIPLES

The magneto-strictive effect (Kashiwaya et al, 1985) is a phenomenon which explains the dependency of magnetic permeability of a ferromagnetic material,



magnetic permeability (in this case, $\sigma_1=\sigma$, $\sigma_2=0$).

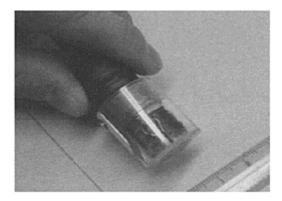


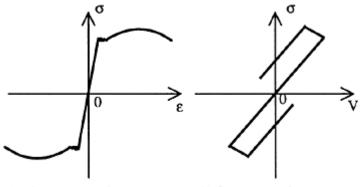
Photo 1. Magnetic anisotropy sensor.

such as steel, on the state of stress. Figure 1 shows the principle of the magnetic stress measurement. When a specimen is subject to the principal stresses σ_1 and σ_2 , ($\sigma_1 > \sigma_2$), magnetic permeability in the respective direction become μ_1 and μ_2 ($\mu_1 > \mu_2$) (Sakai and Tamura, 2000). This anisotropy in magnetic permeability is measured by the sensor shown in Photo 1. The sensor is made of an excitation coil and a detection coil. As controlled current travels through the excitation coil, it produces the corresponding magnetic flux. The flux travels mostly from E_1 to E_2 directly, however, some of it travels along the direction shown in Figure 1 because of the magnetic anisotropy caused by the state of stress. This induces the magnetic flux in the detection core, resulting in current and voltage that can be measured. Because the magnetic anisotropy ($\mu_1-\mu_2$) is proportional to the stress difference ($\sigma_1-\sigma_2$), one obtains the relation

 $V=M(\sigma_1-\sigma_2)$

(1)

where V is the voltage in the detection core and M is a constant determined by excitation condition and magnetic characteristic of the material. The maximum and minimum voltages are obtained from the directions of maximum and minimum principal stress, respectively. This fact enables complete determination of stress tensor of the surface of steel, when combined with proper treatment of stress boundary conditions (Abuku et al, 1986).



(a) Sress v.s strain

(b) Sress v.s voltage

Figure 2. Typical nonlinear relationship among stress, strain and voltage.



Photo 2. Measurement on site.

It is also known (Fujii et al, 1999) that the proportionality between principal stress difference and magnetic permeability difference does not hold during plastic deformation. As a typical relation is illustrated in Figure 2, it is assumed in the context of this paper that the proportionality in the elastic stress range and corresponding constant M determined for standard samples of SS400, are assumed unless otherwise specified.

Thus, this method enables measurement of stress tensor on surfaces of steel, for example, non-destructively, easily, and economically Photo 2 shows measurement operation on site for the case of H-beam, as an example.

3 STRESS ON STRAIGHT H-BEAM

The magnetic anisotropy sensor used in this study has an approximate diameter of 20 mm and measures the average stress in the thickness of 0.23 mm on steel surface. This sensor was first used to identify initial state of stress at the center of four H-beams (SS400, Size: 200*200*12*20) having the length of 2 m. They are named S1 (straight beam), R6000 (initially straight and to be bent with 6000 mm curvature later), and so on. The measurement was conducted in such a way that a complete state of stress was determined at

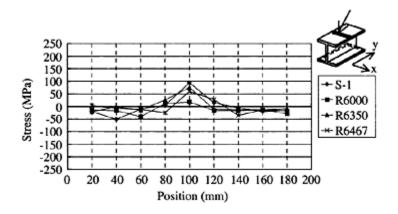


Figure 3. Measured stress on the upper flange.

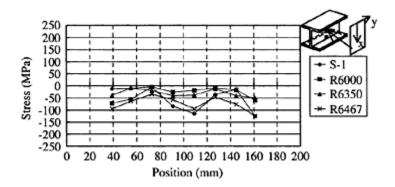


Figure 4. Measured stress on the webb.

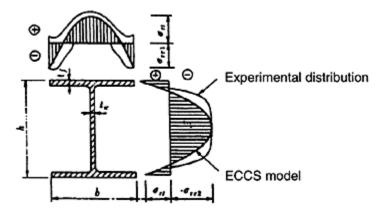


Figure 5. Typical residual stress distribution obtained by conventional method.

10 points (2 cm apart) along the center line of the beams are shown for the upper flange (Figure 3) and webb (Figure 4), respectively. These data are processed using the calibration results for standard SS400 material. It is noted that the stress distribution on the flange surface has a peak at the center where the tensile stress in the order of 50 to 100 MPa are present as residual stress. The stress at both ends goes into compression of less than 50 MPa.

This trend is identical for all four beams. As for the webb surface, the stress in the order of 100 MPa or less are present as compression over the entire face. The peak at the center is recognizable if watched carefully.

The significant findings here are that (1) distribution of residual stress has certain shape which seems to be a unique one for this H-beam and (2) the order of residual stress is considerably high with respect to the level of yield stress of standard steel. This, however, has been widely recognized fact in the field of steel structure. The typical distributions of residual stress for roll-formed H-beams are shown in Figure 5 from the code book (JSCE, 1987). The shape of stress distribution and its order are almost identical with what was measured by the magnetic anisotropy sensor.

4 STRESS ON CURVED H-BEAM

The three beams (R6000, R6350, R6467) were then bent to their respective curvatures by the bending machine shown in Photo 3. This machine takes in an H-beam from one end, pushes it into another end where the final arm supporting the beam moves forward, forcibly bending the beam to a specified curvature under normal temperature.

The stresses were then measured for three beams at the same position. The results from the beam R6000 are shown for Figures 6, 7, 8 and 9, respectively. The stresses on the upper flange face (the convex side) had a tensile stress peak at the center before bending, but now the peak stress at the center shifted into compression. In fact the entire

stress distribution is now in compression in the order of 50 to 150 MPa. On the contrary, the stresses on the lower flange surface (the concave side) shifted into tension also in the order of 50 to nearly 200 MPa.

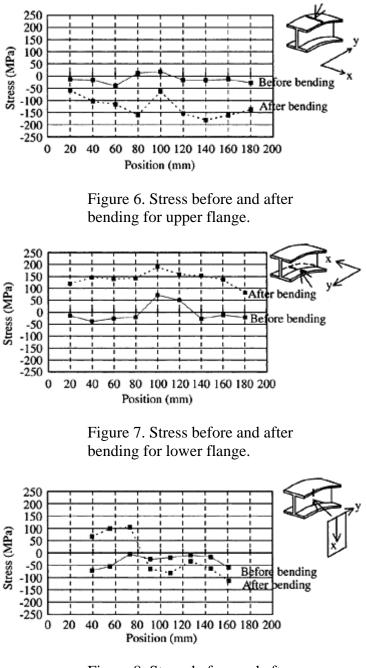
The stresses on the right webb surface shifted into tension over the top half and into compression on the bottom half. The symmetric behavior is observed for the left webb surface.

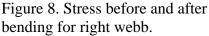
An analysis is needed to understand these behaviors of surface stresses on H-beams before and after the bending procedure with the help from the illustrations shown in Figure 10, which shows the expected paths of a stress-strain and a stress-voltage curves for the upper flange surface, as an example. First, the state of stress is at point A which shows the initial residual stress state while the beam is still straight. As the beam goes into the bending machine, the stress increases initially in an elastic range and then yielding occurs, finally reaching to the point B, where the bending procedure terminates.

As the beam gets out of the machine, the immediate elastic stress kick-back occurs bringing the state of stress from point B to point C. The shape, amount of plastic deformation over the height of the beam, curvature, etc. all contribute to the final state of stress on the upper flange surface which is on the COMPRESSIVE side. The state of stress of H-beams is at the point C when transported to tunnel site. As a beam is put in place around a tunnel face, an insitu loading onto the beam may bring the state of stress towards



Photo 3. Bending machine and Hbeam.





point D or back toward point B depending on a deformation mechanism around the tunnel excavation.

Figure 11 shows contours of stress distribution after the bending procedure. The simulation was performed using the ABAQUS/Standard. The trend of

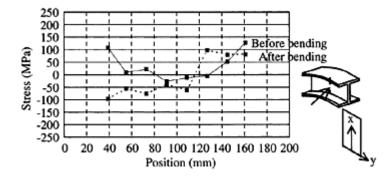
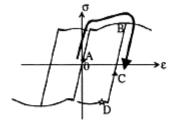
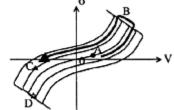


Figure 9. Stress before and after bending for left webb.





 (a) A path on a stress-strain curve

(b) A path on a stress-voltage curve

Figure 10. Paths of stress, strain and voltage during bending.

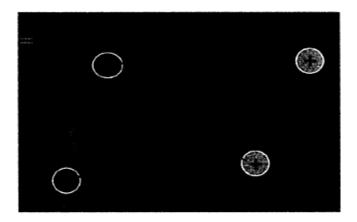


Figure 11. Stress distribution computed by ABAQUS/ Standard using Tresca failure criterion.

residual stress distribution was identical to those obtained from the magnetic stress sensor.

5 LOADING AND CUT TEST

The findings made thus far indicate that an H-beam is subject to considerably high residual stresses caused firstly in a curing process and secondly in a bending procedure. In order to confirm the state of residual stress distribution from a different angle, a simple loading test was conducted. The experimental apparatus is shown in Figure 12.

A bent H-beam, simply supported, was placed in a loading machine in which two point forces are applied

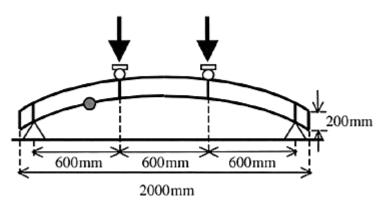


Figure 12. Loading test configuration.

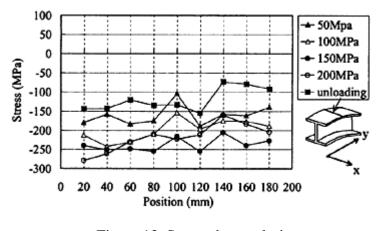


Figure 13. Stress change during loading.

such that the central section of 60 cm would be subject to a constant bending moment. Stresses, strains and displacement at some points were measured step-wise for increasing loads. The loading was arranged such that the stress on the flange would be 0, 50, 150, and 200 MPa, respectively, assuming an elastic response of the beam. The results of stress measurement by the magnetic sensor are shown in Figure 13 for each level of loading. As for the stress at the center of the upper flange, the initial reading was lost. However, the following readings show that the stress values corresponded nearly linearly against loading for the first three steps. Namely, the stresses were about 100, 150 and 200 MPa in compression while the assumed elastic loading should have resulted in 50, 100 and 150 MPa in compression. The 50 MPa difference here is attributed to the initial residual stress which were around 50 MPa. The approximate linearity, however, is lost as the final loading step is given. It is noticed that even though the same loading increment was given, the stress at the center for the upper flange did not increase at all. This in fact is the indication of the onset of plastic deformation. As shown in Figure 2, the voltage measured by the sensor starts to drop as a material goes into the plastic state. Because the state of stress for the upper flange was already in compression, as indicated in Figure 10 as point C, the loading scheme employed in this test brought the upper flange into the yielding state in compression earlier than an

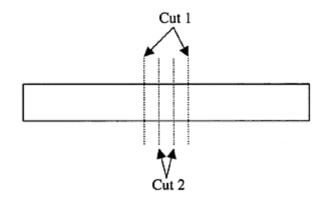


Figure 14. Process of cutting the beam into 5 pieces.

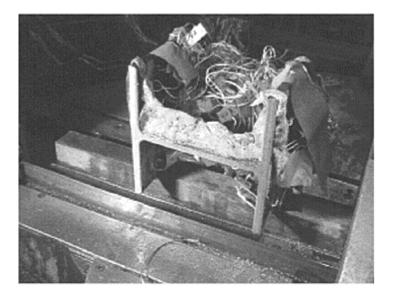


Photo 4. Specimen after Cut 1.

initially anticipated elastic behavior. Considering the fact that the level of initial residual stress was around 50 MPa in compression at the center of the upper flange, all numbers come to fit consistently. That is that when the loading test started, the stress at the center of the upper flange was already 50 MPa in compression.

While the loading was increased to the third level, which would have produced 150 MPa if the beam had reacted elastically from 0 stress, the actual stress was already around 200 MPa in compression. As the final 50 MPa increment was given, the stress at the center reached the yielding stress undergoing plastic deformation while showing little change in the magnetic sensor reading.

One of the curved beam was then cut into three pieces at first (Cut 1). Then the central piece was further cut into three slices (Cut 2), as shown in Figure 14 and Photos 4 and 5. Stress reading by the magnetic sensor was made before and after the Cut 1. Also, strains were monitored at the locations during the Cut 1. Some strain gauges were added during the Cut 2 as well along the center line of the specimen.

The results shown in Figures 15 and 16 show that after loading tests were completed, the residual stresses in the order of 50 MPa or more still existed on the

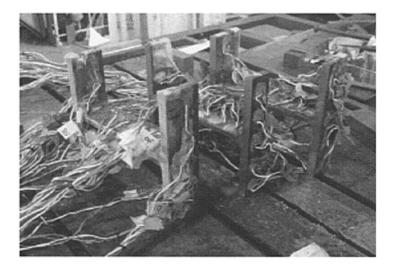


Photo 5. Specimen after Cut 2.

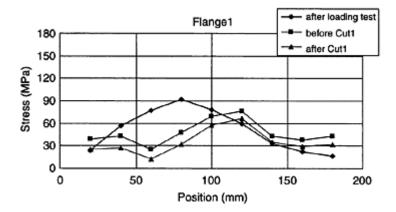


Figure 15. Equivalent stresses on the flange surface released during the cut processes (measured by magnetic sensor).

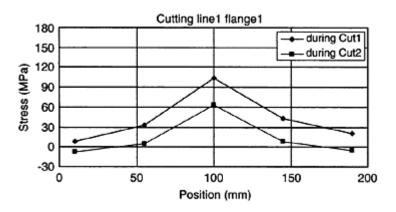


Figure 16. Equivalent stresses on the flange surface released during the cut processes (measured by strain gauges).

flange surfaces. The estimated values of residual stresses by the magnetic sensor can readily be confirmed by the released stress computed by measured strains.

6 FIELD MEASUREMENT EXAMPLE-1

The applicability of the magnetic anisotropy sensor was tested at the Nagata Tunnel constructed by the

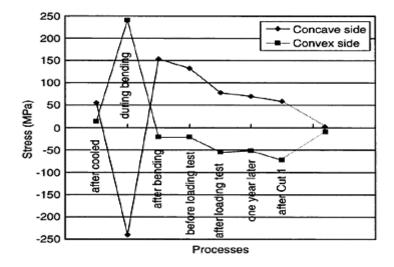


Figure 17. Typical stress history for an H beam on flange surfaces.

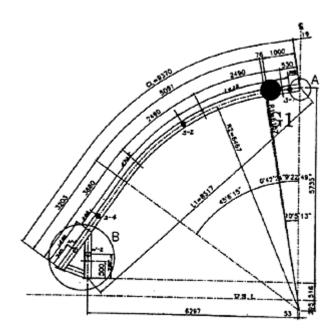
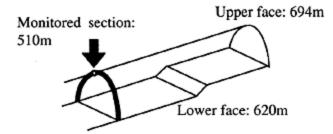


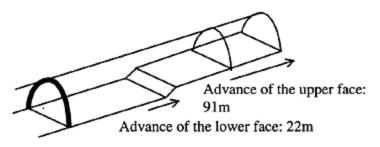
Figure 18. Tunnel cross section.

Hanshin Expressway Public Corporation in the city of Kobe, Japan. The measurements were conducted twice at the cross section 510 m from the tunnel portal where strain gauges had been installed for monitoring. Figure 18 shows the cross section and the measurement position indicated as G1. The first and second readings were taken while the tunnel face advanced some distance. Figure 18 shows the timings of measurement with regards to the progress of construction. The measurement results are shown in Figure 20. The stress distribution over the flange surface has similar shape as those observed before. The initial stress at the center was around 50 MPa in compression. By comparing this value with an extra line shown in Figure 20 as the probable stress distribution of the beam, one notices that the increment of compressive



*Figures indicated distance from the portal.

(a) First measurement.



(b) Second measurement.

Figure 19. Position of tunnel faces.

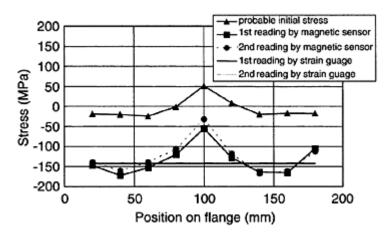


Figure 20. The results of stress measurements.

stress of about 100 MPa could have acted here during tunnel construction. Overall change in stress distribution between two measurements, are relatively small. This is consistent with the results of strain guage readings shown as straight lines. As this measurement section was chosen as a representative section where not much stress change was occurring, the measurement results from both the magnetic sensor and strain gauges supported the initial expectation with reasonably accuracy.

7 FIELD MEASUREMENT EXAMPLE-2

The applicability of the magnetic anisotropy sensor was tested at the Kitasuma Tunnel constructed by the Hanshin Expressway Public Corporation in the city of Kobe, Japan. The measurements were conducted three

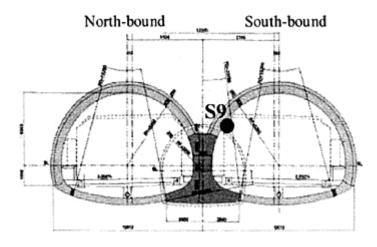


Figure 21. Tunnel cross section for Kitasuma tunnel.

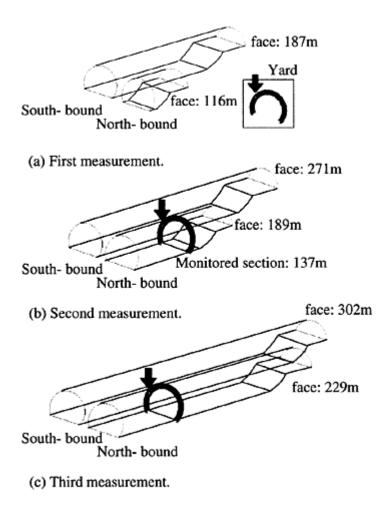


Figure 22. Position of tunnel faces for 3 measurement timings.

times. Firstly, the initial stresses were read for a bent H-beam before it was put in place. The second and third readings were taken after it was put in a particular section of the tunnel. Figure 21 shows the cross section and the measurement position. Figure 22 shows the timings of measurement with regards to the progress of construction. Before the second reading, the surface for the measurement was cleaned of shotcrete and smoothened by sand paper to assure workability for the sensor.

The measurement results are shown in Figure 23. The stress distribution over the flange surface has

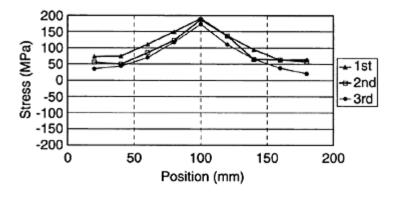


Figure 23. Results of field measurement of stress.

Table 1. Average stress changes obtained from the magnetic anisotropy sensor and strain gauges (Unit: MPa).

Measurement timing	Magnetic sensor	Strain guage
Between		
1st and 2nd measurements	14.8	32.6
Between		
1st and 3rd measurements	31.5	40.2

similar shape as those observed before. The initial stress at the center was around 180 MPa in tension since this is on a concave side of the beam. The state of stress on this surface then experiences compressive stress increment over entire width of the flange.

For example the increment on the edge is in the order of 50 MPa. It is of interest to know that the H-beam put in place is usually subject to an increase in axial force, which actually is happening in this case as well. However, because of the distribution of initial residual stress, the actual stresses on the flange surface are still in tension.

In order to compare the readings from the magnetic anisotropy sensor with those from the conventional method using strain gauges, the average changes in surface stress were computed as shown in Table 1. The stresses from the strain gauges were computed by linearly extrapolating stress values at the point of strain gauges which were actually installed on the surface of the webb. The results were in good agreement considering the expected fluctuation of the magnetic anisotropy sensor which has been identified thus far as 10 to 20 MPa from previous application to bridges, etc.

8 CONCLUSION

A magnetic anisotropy sensor was used for non-destructive measurement of stress on surfaces of H-beams used as structural elements in tunnel support systems. The sensor is built on the principle of the magneto-strictive effect in which changes in magnetic permeability due to deformation of a ferromagnetic material, such as steel, are measured and converted to absolute values of stresses existing on the surface of the material. Proper treatment of boundary conditions allows determination of stress tensor completely on surface of H-beam flanges, with average error of 10 to 20 MPa.

A series of stress measurement was conducted for H-beams in NATM tunnels. The results obtained from the field stress measurement were in full accordance with the findings made from the previous series of measurement in a lab. The sensor used in this study is small, easy to carry around even into a tunnel, for example, and allows direct, nondestructive measurement of absolute values of stresses on surface of steel.

The work presented in this paper shows the illustrative application of this technology for the case of H-beam used in tunnels; however, the use of this sensor is not limited to H-beams and it can be applied in any situation where surfaces of steel, in the forms of beams, plates, pipes, walls, etc. are exposed for human access. Not only a direct use of the results of stress measurement is beneficial in numerous civil and mining engineering applications, but also a secondary use of the information for interpretation of deformational behavior of structures in concern, in this case a tunnel, is also possible.

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Support performance control in large underground caverns using instrumentation and field monitoring

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ABSTRACT: A careful monitoring program at Masjed Soleiman underground powerhouse enabled determination of hazardous areas around the main cavern and was used as a controlling tool to check the adequacy of a new support program. Preliminary support design in centerline and downstream side of the powerhouse cavern showed to be not enough to maintain the integrity and stability of the rock mass. The adequacy of the additional ground anchors was proved by recording a decreasing trend in rock mass displacements using monitoring instruments. The upstream side did not require further support according to the monitoring results but was additionally supported for higher safety factor. The powerhouse cavern and other peripheral excavations are now finished and final concrete structure is under execution. The installation of the power generator units will finish by the end of year 2004.

1 INTRODUCTION

Most of underground structures are built in grounds where without some means of improvement can not maintain their stability. These reinforcement operations range widely from compaction, drainage and grouting of soft soils to installation of fully grouted bolts and applying shotcrete in jointed and blocky rock masses. The amount of ground improvement measures which is really adequate for each job is a challenging question in front of the engineers. A heavy reinforcing system can assure safety but is often not economical. On the other hand, insufficient measures will not assure the safety of the structures therefore having an accurate ground improvement program always is a question to be answered.

In recent decades with advances in other branches of science and technology, specially electronics and mechanics, very accurate instruments are developed which are frequently used in geomechanical projects. These instruments have helped a lot to determine the exact performance of the surface and underground structures as well as the reinforcement and improvement aids. With continuous monitoring of the whole system, one can pin point the problematic locations and quantitatively determine the sufficient improvement measures to provide the required safety factor.

This paper has focused on an example of underground hydroelectric power plant project in Iran which has benefited from a careful monitoring program. This will show how a continuous interaction between obtained data from the monitoring team and prompt response by the engineering and contractor teams have helped to overcome some instability problems by applying an economical reinforcing program.

2 MASJED-SOLEIMAN HYDROELECTRIC POWER PLANT

Masjed-Soleiman Hydroelectric Power Plant (HEPP) is built in two phases with total capacity of 2000 MW on Karun River in Iran. The rock mass consists of compact Conglomerates, Siltstone, Sandstone and Claystone layers crossed by widely spaced joints. The dimensions of the powerhouse cavern are 266 m in length, 30 m in width and 50 m in height. Transformer cavern is located to the right side of the powerhouse cavern at a higher elevation as shown in figure 1. The layers dip at about 25 degrees towards the upstream side. Day lighting Claystone in the roof and the walls of the caverns with low frictional and mechanical properties has resulted in some instability problems.

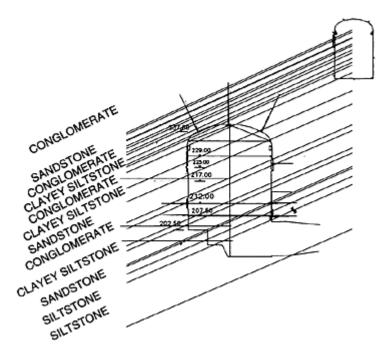


Figure 1. Underground power station scheme and rock mass structure.

	Conglomerate	Sandstone	Claystone
Young's Modulus (MPa)	15	7	6
Poisson's ratio	0.2	0.2	0.25
Cohesion (MPa)	2.28	1.67	0.73
Friction Angle (Degree)	43	38	30

Table 1. Rock type and its mechanical properties.

3 GEOMECHANICAL PROPERTIES OF THE ROCK MASS

Based on laboratory and field tests performed the following parameters are reported by Stabel (2002). Low mechanical properties of the Claystone rock especially when absorbing water results in further drop in the values and results in instability at the roof.

Excessive rock mass displacement which causes shotcrete cracking and bolt failure could be seen in some parts of the roof especially in the centerline and downstream side of the powerhouse cavern.

4 MONITORING PROGRAM IN THE PROJECT

To control the stability of the structure while excavating the caverns in stages, a comprehensive instrumentation program was proposed and implemented for the caverns which consisted of 200 instruments

Table 2. Monitoring stations at phase 2 of the
powerhouse and transformer caverns.

Chainage of the monitoring stations (measured from the beginning of the cavern)						
Powerhouse cavern	8	25	43	75	93	107
Transformer cavern	0	28	58	78	100	110

Table 3. Type of instruments in the powerhouse and transformer caverns.

4 point, 30 m/15 m long borehole extensometers.	50 t and 200 t load cells and 2 MPa pressure cells

4 point, 15 m long borehole extensometers, 50 t load cells and 2 MPa pressure cells

including borehole extensioneters, load cells and pressure cells distributed in the caverns according to tables 2 and 3.

Some of the monitoring results are depicted in figures 2–4. The upper graph is rock mass displacement recorded by borehole extensometer, the lower graph is the corresponding load cell results with time. As shown in figure 2, which corresponds to monitoring station at chainage 21 at downstream, the rock mass has kept moving although the reinforcement program had fully been applied according to the initial design. This is also the case for chainage 71, centerline as shown in figure 3. As noted in figure 1, this extra amount of rock movement in downstream is due to the presence of soft Claystone layer with potential of swelling. However, at the upstream side and to some extends, at the centerline, the rock mass movement is much less and in many cases the rock has stopped moving after a while. Such an example is depicted in figure 4 for chainage 71, upstream side.

Increasing trend of the rock movement has resulted in an increasing trend of load in the load cells. At a stage, the load in the bolts had increased to a level very close to the yield capacity of the bolts. The bolt containing this load cell was unloaded and the load cell was installed again to be able to detect further increase in bolt load. This increasing trend of displacement and load in centerline and downstream side together with a local rock fall alarmed the engineer that more support pressure than what was anticipated earlier is required to assure long term stability of the cavern.

The additional support program consisted of 15 and 20 meters long tendons (Double Corrosion Protected, DCP) with 60 ton working capacity. This system was gradually applied to the whole roof of the powerhouse cavern as shown in figure 5.

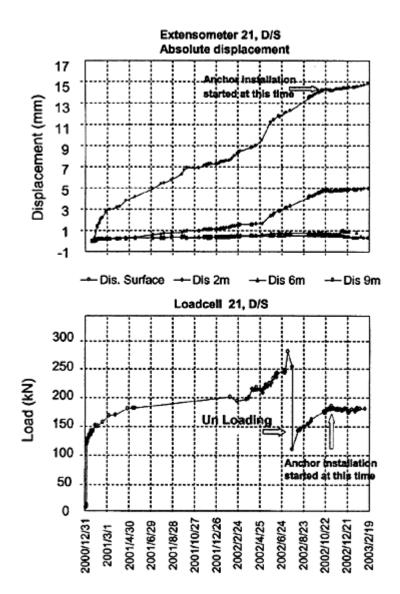


Figure 2. Displacement (upper) and load increase (lower) at chainage 21 m, downstream.

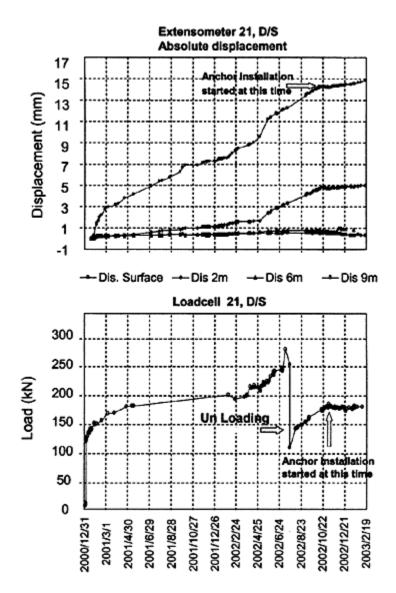


Figure 3. Displacement (upper) and 3 increase (lower) at chainage 71 m, centerline.

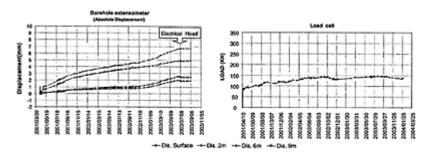


Figure 4. Displacement (upper) and load increase (lower) at chainage 71 m, upstream.

Start and the end of ground anchor installation in the vicinity of the extensometer at chainages 21 m and 71 m are shown by arrows on the previous figures. This remedial work stopped the increasing trend of rock movement gradually according to these figures. What happened at the roof in terms of rock pressure and reinforcement pressure is summarized in table 4. To calculate the roof and support pressure, the suggestions of Barton (1974) and Hoek (1999) are used respectively.

As can be seen from table 4, the support pressure from reinforcements before adding complementary reinforcement (0.192 MPa) is less than what is applied by the roof at centerline and downstream side (0.225 MPa) which explains increasing trend of the recorded rock mass displacements by the instruments. After applying extra pressure by additional support, the reinforcement pressure becomes more than the roof pressure (0.288 MPa) and results in the roof stability. The initial support pressure in upstream side was originally more than the roof pressure which explains the decreasing trend of rock movements in these sides. The applied additional support proved to be not necessary for upstream.

The above mentioned example illustrates how we can make advantage of a reliable monitoring program to control the rock mass stability and to feed the

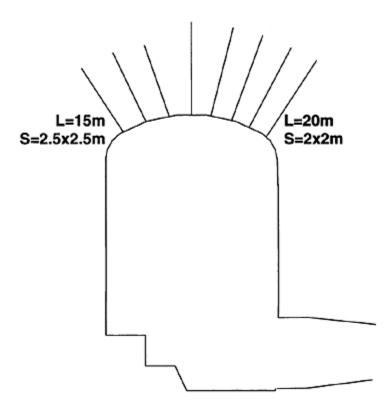


Figure 5. Additional support scheme.

required information to the design team for an optimal final ground support design.

To summarize the rock movement profile of the cavern roof and the depth of the movements and corresponding load increase in load cells, table 5 is presented.

To get a general idea about the situation of this cavern with respect to the other cases around the world, the rock displacement is overlaid in the graph presented by Barton (1999) for cavern displacement around the world (figure 6).

Barton has gathered the information of a number of the caverns around the world in terms of the recorded rock mass movement and the quality of the media and has come up with the following empirical relation

$$\Delta(mm) = \frac{span(m)}{Q}$$

in which Q is rock quality index (NGI system), span is the width of the cavern in meters and Δ in millimeters is the rock movement. Plotting the recorded rock mass displacements in figure 6 by Barton shows that the displacements at this cavern conform to the range

Location	Displacement (mm)	Depth of the rock movement (m)	Load increase in the load cells (kN)
Downstream	24	9–12	230
Center line	10	6–7	120
Upstream	4.6	5–6	50

Table 5. Rock movement profile of the powerhouse cavern roof.

Table 4. Summary of the roof and support pressure before and after additional reinforcement.

			Reinforcement pressure (MPa)		
Reinforceme cavern	ent system in the powerhouse	Roof pressure (by Q)	Before additional support	After additional support	
Powerhouse cavern	D/S- ϕ 28 mm, L=6 & 12 m, spacing 1.75× ϕ 40 mm, L=20 m and 2×2 m spacing	0.225	0.192	0.288	
	C/L- ϕ 28 mm, L=6 & 12 m, spacing 1.75× ϕ 40 mm, 1.75, L=20 m and 2×2 m spacing	0.225	0.192	0.288	
	U/S- φ 28 mm, _{L=6 & 12 m} , spacing 1.75×1.75, Tendon φ 40 mm, _{L=15} m and 2.5×2.5 m spacing	0.05	0.192	0.253	
Transformer	φ 28 mm, _{L=6} & 12 m, spacing 1.75×1.75 m	0.05	0.073		

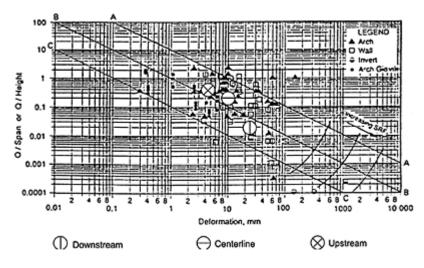


Figure 6. Comparison of the recorded rock movement with other cases in the world.

of what have been recorded in other projects around the world.

5 CONCLUSIONS

Monitoring program at extension phase of Masjed-Soleiman underground power plant is a typical example of a reliable monitoring program to control the amount and performance of a ground improvement technique to assure stability requirements for an underground structure. The continuous rock mass movement results recorded from borehole extensometers necessitated additional support installation at the downstream side which later on by installation of more reinforcement proved to be adequate by observing a decreasing trend in movements. On the other hand, the rock movement trend in upstream side of the cavern indicated no further support requirement, the fact that was also confirmed through experimental load calculations for the roof and the supports.

ACKNOWLEDGEMENTS

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Rock mass classifications and complementary analyses of use in tunnel design

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ABSTRACT: The paper briefly describes common reasons for rock failure and support installation in tunnel construction. Where tunnelling conditions cannot be calibrated to local experience or classification systems, the need for a thorough site-specific investigation of rock conditions is emphasized. This investigation should permit recognition and analysis of the full range of potential rock failure modes. The use of rock mass classifications systems in blocky rock conditions is discussed, as well as the need for complementary analyses and support criteria when developing tunnel designs in rock masses where there is a possibility of encountering heavily fractured or soil-like zones, overstress and/or swell/slake behaviour. The need for analyses that complement rock mass classifications is underlined by reference to a recently completed set of tunnels excavated in dolomite, siltstone and shale for the Neutrinos at Main Injector (NuMI) Project.

1 INTRODUCTION

Tunnelling presents a design team with some unique challenges not commonly encountered in other branches of engineering. In particular, the properties and loading of the construction material (the ground) are highly variable and this variability cannot be defined with accuracy along the length tunnel. This uncertainty as to rock conditions along an alignment creates a fundamental challenge to the engineer because there will be inadequate data to support a structural analysis, able to model the opening in its "natural complexity."

In more complicated geologic settings, the lack of reliable engineering data will severely limit the value of a comprehensive design model. Too much guesswork is needed to satisfy model input relative to the definition of material and loading parameters.

The dearth of reliable geotechnical data largely explains why many tunnel designers continue to rely heavily upon the use of empirical classifications, indices, rules-of-thumb

and simple rock mechanics models as a basis of design. In nearly all instances, these design tools have been developed from and/or calibrated to case history precedent.

Given the importance of precedence in many tunnel designs, it is important for the practitioner to gain an appreciation for the various types of adverse rock mass behaviour that can occur either singly or in combination underground. It is also important that the designer understand the impacts that such adverse behaviours can have on stability, support requirements and on mining productivity, Having gained an appreciation of the causes and impacts of adverse behaviour, the need for conscientious and systematic investigation, identification, analysis and mitigation during design will become apparent.

Design tools, such as classification systems, based firmly on precedent can help predict adverse behaviours but their application is limited by the inherent biases of the data sets upon which they were formulated. Relying solely on such empirical design tool can leave a project vulnerable to unexpected, unmitigated and potentially serious problems during construction.

2 ROCK CONDITIONS THAT IMPACT EXCAVATION AND SUPPORT

2.1 Overview

Predicting the impacts of rock conditions on a tunnel project and designing appropriate mitigation plans requires a broad consideration of geology, rock mechanics and tunnel construction techniques.

With respect to the adverse impacts of rock conditions, a review of tunnel case histories leads to the identification of five basic types of rock condition that commonly cause instability underground. These conditions are:

- Discontinuity-bounded blocks/wedges,
- Soil-like inclusion/pockets in the rock matrix,
- Overstress at the tunnel boundary,
- Shrink-swell behaviour and
- Water inflow and pressure.

The rock conditions giving rise to such problems are briefly described below.

2.2 Discontinuities

Rock block and wedge fall-out, bounded by natural discontinuities, commonly joints and bedding, are encountered in most rock tunnels. These fall-outs are observed in the arch and sidewalls. Design methodologies have advanced to provide for support design in "pattern-fractured" rock masses.

The ability of the discontinuity-bound blocks and wedges to resist sliding under gravity is reduced as the discontinuity surfaces become smoother. The shear strength of the contact is also reduced through the action of alteration and weathering.

The characteristics of the discontinuities and their associated weathering and alteration are strongly related to the geologic origin, geo-structural history and specific mineral content of the rock unit (Price, 1993). Recognizing the stereotypical fracture, weathering and alteration patterns that are commonly associated with specific rock units is key if the design team is to develop an early appreciation of the likely range of rock mass conditions along an alignment.

2.3 Heavily fractured or soil-like zones

Tunnel failures can occur along seams, lenses, and pockets of heavily fractured or soillike materials encountered within a rock mass. Such zones can be formed by shear deformation combined with the actions of alteration and/or weathering or may be the result of post-formational infilling of rock voids (e.g. karst). Particle size of the fill materials can vary widely from blocky gravels to fine clay. These materials are most susceptible to failure when saturated and/or subjected to the piping effects of water inflow.

2.4 Overstress

In the literature, overstress conditions are most commonly reported in associated with brittle failures (rock burst or popping) occurring at depth in strong rock masses (e.g. Graham, 1976). However, overstress conditions are not confined to deep mining operations, they are also found at shallow depth, where high horizontal stresses and unfavorable tunnel orientations combine to cause failure. Most notably, high horizontal stresses have been noted as causing overstress in glaciated regions such as Buffalo, New York (Nelson, 1984) and adjacent to fjord-deepened valley sides in Norway (Broch, 1984).

Overstress behaviour in ductile rock masses can result in rapid convergence or squeezing. This squeezing action can be sizeable and rapid and can lead to the entrapment of the excavation equipment, as documented in the mining of the Yacambu tunnel (Babendererde and Babendererde, 1996).

2.5 Swelling

Although less common, high levels of deformation and/or support loading have been reported in shrink/ swell-susceptible rock materials exposed to water. International Society for Rock Mechanics (ISRM, 1983) note that swelling may occur in rocks having minerals susceptible to physio-chemical reaction that involves water i.e. rocks containing clay minerals, anhydrite or pyrite. Swelling rocks commonly exhibit low durability and will slake or deteriorate rapidly when exposed to water. If not properly mitigated by avoiding water contact and/or installing substantial structural linings, swell-driven convergence rates can be dramatic. ISRM reports that swelling rates observed in tunnel inverts have attained rates of up to one meter per month.

2.6 Water pressure and inflow

Water pressure and inflows are frequently noted as contributing to instability and, even under stable rock mass conditions, water inflow is often itself cited as causing problems for the tunneling operations where flows exceed roughly 10001/m per 100 m of mined tunnel at the heading (Laughton, 1998).

The La Réunion project (Association Française des Travaux en Souterrain, AFTES, Working Group Number 4, 1994) demonstrates fairly succinctly the impacts that water inflow can have on a mining operation. On this drive, a Tunnel Boring Machine (TBM) was used to drive a tunnel through basalts, sited below the water table. Water inflow reduced TBM advance rates by varying degrees dependent upon both the rate of water inflow and the rock mass conditions at the heading.

In dry, stable rock conditions, an average Advance Rate (AR) of 16.8 meters per day was achieved. The average daily AR decreased by approximately 25 percent when a combination of high water inflow and good rock mass conditions was encountered. Where the heading was subjected to the same high water inflow in poor rock mass conditions production was reduced to well below half that noted in dry, stable conditions. The wet/poor rock mass combination caused significant instability at the face, with water both flushing-out fill material and displacing wedges and blocks.

2.7 Ground support reasons

Surveys of underground rock support show that four of the five types of rock conditions described above are ones that commonly cause problems underground.

In a rock support survey of five Swedish underground projects excavated in granites and metamorphic rock, Brannsfors and Nord (1979), reported that in 99% of the cases where ground support was increased, the need for the increase was attributed to four of the rock mass factors described above. The frequencies with which these factors, acting either singly or in paired-combinations, were cited as causing instability are shown in the Venn diagram in Figure 1. The frequency of each cause is defined as a percentage.

In most instances discontinuity-related features (blocks and wedges) were identified as a cause for instability either acting singly or in combination with water. Equally as significant is the fact that water was noted as a contributory factor to instability in 77% of the instances, but was not cited as the sole cause of instability in any specific case. Just "adding water" to a rock mass can have a major deleterious impact on the overall behaviour of the excavated opening. Given the added negative impact that water can have on mining productivity, the value of mining "in the dry" should not be underestimated within the overall evaluation of the viability of a tunnel project.

Failures through soil-like materials (fill) were commonly associated with the presence of water. Overstress and swell behaviour were not separately noted, but an intact rock/water combination was identified as requiring extra support in just 6% of the case histories. The complete absence of any case where

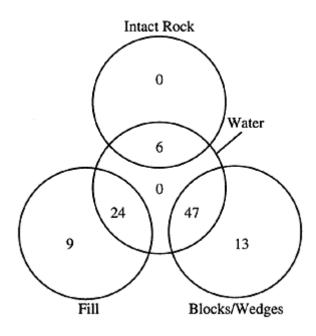


Figure 1. Cause(s) for increased ground support

instability was attributed solely to failure through dry, intact rock may be largely explained by the inherent bias of the projects' data sets towards the excavation of relatively high strength, low clay content rocks mined under low to intermediate stress conditions.

Tunnel surveys conducted in the USA and Japan also identify discontinuities as the leading cause of instability in tunnels constructed for civil engineering purposes. A survey conducted in North America by the Subcommittee on Geotechnical Site Investigations, US National Committee on Tunneling Technology of the National Research Council (USNCTT, 1984) identified discontinuities as the single most common cause of problems and claims. In Japan, Inokuma et al. (1994) noted that over 70 percent of tunnel "trouble spots" were discontinuity-related. This study considered a set of 65 Japanese tunnels, driven in a wide variety of rock types.

Using precedent as a guide, it is clear that a tunnel is more likely to be subject to discontinuity-related problems than to other forms of adverse behaviour. However, the consequences of encountering any of the other types of failure on overall tunneling performance (cost and schedule) can be severe. Hence, although less common, a conscientious engineer should investigate the potential for every type of failure on every tunnel job.

The rock tunnel designer may never know all the answers, but that is no excuse for not asking all the necessary questions or pursuing the appropriate research to answer such questions. Even if absolute answer cannot be found the designer must understand the full gamut or risks that will be run during the excavation process.

3 TUNNEL DESIGN

3.1 Overview

The main design and construction phases of a rock tunnel project are shown in the flowchart in Figure 2 (Lowe, 1993). The flowchart outlines the basic sequence of activities undertaken on a tunnel from alignment selection through the end of construction.

The flowchart provides a framework for the discussion that follows with regard to the early stages of the design, namely, site investigation and alignment, rock mass characterization, selection of excavation methods and means and structural elements. It is these steps in the design that will play a determinant role in the overall success of the underground project.

3.2 Site investigation and alignment

As Muir Wood (1972) lamented "too many site investigations for tunnels comprise a regular pattern of

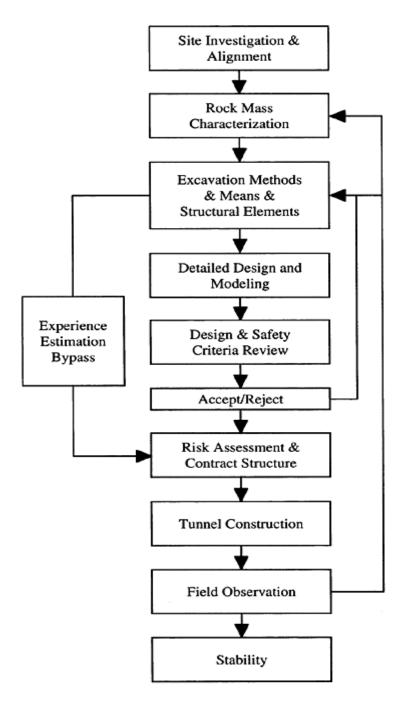


Figure 2. Simplified tunnel design flowchart.

boreholes, a conventional package of tests and a sigh of relief when it is over." This lamentation is an can be regarded as an open acknowledgment that all too often the planning team will not be appropriately equipped (staffed or resourced) to draw upon the geologic and tunneling expertise required during the early planning phase of the project (e.g. literature search, desk and initial field studies). If this is so the site investigation may be poorly focused and fail to identify such problems as are described above. If these problems are not recognized during the investigation period the ability of the designer and builder to properly mitigate them will likely also be reduced.

The early involvement of experienced geologists and tunnel engineers is key during the development of the investigative campaign and selection of an alignment. Not only will general issues relative to the subsurface conditions, presence of unconformities and proximity to other structures need to be addressed, but key characteristics of the rock mass will also need to be identified so that the investigation can be tailored to specific project needs. Without an understanding of the rock characteristics likely to cause problems underground, it will be difficult, if not impossible for a planner to scope-out, prioritize and sequence the investigation and identify relevant tests. Rock mass parameters and classifications that are used to characterize the rock mass and in the design of rock support are discussed below.

3.3 Rock mass characterization

For each discrete zone along a tunnel, the engineer needs to develop a representative rock mass characterization that will allow for the identification of a ground support system(s) and bring in to evidence the likelihood of all types of adverse rock mass behaviour. Dearman (1988) suggested that, in general, separate zones are required when the geo-structural regime changes, when the rock unit changes; and when major fault and fracture zones are anticipated at tunnel depth.

To facilitate this process, the engineer may rely upon the use of classification systems. These classification systems contain parameters that can help recognize the potential for one or more of the problems described above. These systems may also help identify key shared engineering characteristics that can be used for simple modeling and support the selection of appropriate mining methods and means.

Some of the more common classification systems and their uses in industry are briefly outlined below.

3.3.1 Terzaghi rock conditions

Terzaghi (1946) was one of the first practitioners to propose a rock mass classification system that could be used directly as a basis for identifying rock support requirements. Terzaghi described nine distinct types of rock condition and behaviour and developed rock load profiles for them. The descriptions of the rock conditions associated with each of the nine observed types of rock condition (1–9) are listed below:

• Hard and Intact (1)

- Hard stratified or schistose (2)
- Massive, moderately jointed (3)
- Moderately blocky and seamy (4)

- Very blocky and seamy (5)
- Completely crushed but chemically intact (6)
- Squeezing rock at moderate depth (7)
- Squeezing rock at great depth (8)
- Swelling rock (9)

Terzaghi notes that the tunnel rock loads associated with the blocky rock conditions can be halved if the tunnel is sited above the water table.

Although the support method (steel arch) for which Terzaghi's system was primarily designed has been largely superseded, the classification is still commonly referenced. Terzaghi's description of the rock provides directly for a prediction of how it will behave and also defines design loading. However, the designer is offered no clear guidance on how such rock conditions can be predicted during the site investigation phase of a tunnel project. The number-based classifications described below provide guidance on how support designs can be developed using site investigation data.

3.3.2 Rock Quality Designation (RQD)

RQD (Deere & Deere, 1989) is a "modified core recovery" index value originally devised to differentiate between recovered cores that were sound versus those that included sections of heavy fracturing and lengths of altered or weathered material. The index can be used directly to design bolt support in blocky rock masses.

RQD has been in use as a stand-alone design index for 30 years and has also been incorporated into two other widely used numerical rock mass classification systems, namely the Q-System (Barton, 1976) and the Rock Mass Rating (Bieniawski, 1974).

3.3.3 Multi-parameter classifications

The Q and RMR classification systems use multiple parameters that can be obtained from a site investigation campaign. The parameters used in each system can be related to one or more of the factors that cause tunnel instability. The parameter weightings are derived primarily from the back analysis of suites of blasted tunnel case histories. The weighted values for each parameter are added (RMR) or multiplied (Q) and the results used to provide for an estimate of tunnel support needs.

In the Q-system RQD is used in conjunction with a joint set number (J_n) denominator to represent a discontinuity density value or a crude measure of block size (RQD/J_n) . Discontinuity roughness (J_r) is used with an alteration denominator (J_a) to account for the frictional characteristics of the discontinuity surfaces (J_r/J_a) . These two factors that accounting for block size and inter-block shear strength, are complemented by the use of an "active" stress factor that combines a water pressure numerator (J_w) with a Stress Reduction Factor (SRF) denominator. The SRF is a stress parameter derived from a consideration of the in situ stress state and rock material properties. The water numerator accounts for the impact of water under pressure acting within the rock mass. This index can be used directly in the design of bolt and shotcrete supported openings but, more importantly, the individual parameters can also be used to identify the potential for the occurrence of individual failure modes that may require the application of ground support systems other than those proposed within the scope of the Q-system. In the RMR system, points are allocated and added to arrive at a basic rating. Eighty out of the one hundred point maximum rating is attributed directly to discontinuity parameters, with 50 allocated to discontinuity "density" parameters (RQD and fracture spacing), and 30 points to discontinuity conditions. The rating is adjusted as a function of the orientation of the tunnel axis relative to the more critical planes of weakness of the rock mass. Up to 12 points may be deducted as a function of discontinuity orientation relative to the tunnel alignment. As for the other classification systems, the RMR can be used directly to design ground support requirements.

3.4 The role of number classifications

Since the number classifications were first formulated in the 1970's, tunnel excavation and support technologies have improved significantly and many more kilometers of tunnels have been mined in a wide range of rock conditions. However, despite these developments the basic RQD, RMR and Q-Systems parameters and system weightings have remained unchanged. No comprehensive re-evaluation of the systems has been undertaken that reflects the changes in technology or includes experience in more varied rock conditions. It is suggested that the classifications be updated and expanded to include the use of newer excavation technologies and more diverse rock conditions.

As part of these overhauls it is also suggested that access be provided to the "founding" data sets that will serve as the basis for back-analysis and parameter selection and weighting. Direct access to the case history data sets would provide the user with better insight in to the classification process and provide for individual matching of new projects to specific case history sets.

The need to access the detail for individual case history data sets is underlined by Deere and Deere (1989). In their review they emphasize the need to pay attention to the details of a site. In particular the authors note "perhaps the most common complaint was not against the RQD *per se* but the manner in which it is often used in design as the sole parameter without considering the geologic details and the overall geologic evaluation of the site." The comment is well taken and underlines the need for the designer to use caution in calculating and applying a classification number unless they are confident that a classification system is indeed appropriate to apply on a particular site.

Rock mass classification systems can serve key roles in the planning and design process, but only if they are applied correctly. The user must have a clear understanding of the strengths and weaknesses of a system to ensure that the geologic and construction parameters of importance will get proper consideration within a given classification system.

3.5 Excavation methods and means

After initiation of site investigation, detailed consideration should be given to the selection of excavation methods and means. In developing the best design, end-user needs (excavation size, shape, orientation) may need to be adjusted to take account of the constraints of the rock mass. The practicality of using various construction methods and means should also be reviewed to make sure that they are "ground-compatible" and affordable.

Parameters that influence the structural behavior of the tunnel and need to be integrated during design are shown in Figure 3.

In the design process shown in Figure 1, attention is drawn to the presence of an "Experience Bypass" that allows the designer to limit or forego detailed design by referencing experience gained from relevant case histories.

Case history data has always played an important, if not dominant, role in the design and construction of excavations in rock. Underground, the engineer works with natural materials that are never fully characterized. In more complex, less-investigated rock masses, site-specific geotechnical data may not provide more than "clues" as to the types of ground behaviour to be encountered en route. Site investigation data should always be supplemented, wherever possible, with relevant information and experience gained from other job sites. Tunneling expectations are most realistic when they can be directly calibrated to the experience of others. The larger and more relevant the precedent brought to bear in setting expectations, the better the

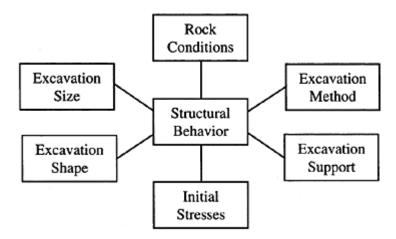


Figure 3. Factors influencing the structural behaviour of a tunnel, after Sutcliffe et al., 1990.

design decisions are likely to be and ultimately the greater the chances of successful tunnelling.

Given the unquestioned value of past experience in the design, effort should be expended in researching and referencing individual case histories reported in the literature as well as assessing the relative quality of the rock mass by reference to classification systems. Some of the better-documented compilations of case histories are those of Cecil (1970), Sinha (1986), AFTES (1994), and USNTTC (1988).

4 CLASSIFICATIONS IN PRACTICE

4.1 Case history description

The tunnel case history referenced is that of the Neutrinos at Main Injector (NuMI) project, built underground on the Fermi National Accelerator Laboratory Campus some 50 km west of Chicago, Illinois, USA. At the site, a combined length of approximately 1500 m of tunnels lies at depths of 20 to 100 m below ground level. The rock tunnels were mined in sub-horizontally bedded dolomite, siltstone and shale rock units that were overlain by approximately 20 m of glacial soils.

The NuMI tunnels house particle beamline equipment aligned to transfer a beam of sub-nucleic particles from an existing particle accelerator, the Main Injector, to a site in northern Minnesota. The tunnel enclosures are inclined at slopes from 6% to 16%. The spans of the NuMI excavation vary from 2 to 10 m. Two 7 m diameter vertical, circular shafts provide access to the facility.

4.2 The rock mass "as investigated"

At the earliest stage of the project, a literature review was conducted to identify relevant tunneling case histories with site conditions that closely matched those of the NuMI site. As a result of these reviews, the designer, Montgomery Watson Harza (MWH), identified several case histories within a 100 km radius of the campus, including sites in the Chicago and Milwaukee metropolitan areas. A number of quarries and room-and-pillar mining operations were also identified. All these excavations had encountered one or more rock units that were to be excavated on the NuMI tunnels. Rock mass characterization, design, contract, construction and cost information was collected from many of these projects. Where possible, field visits were conducted with the owner and designer to observe conditions in situ. The NuMI Project's expectations for rock conditions; ground support, water inflow levels and end-user requirements were all "sanity-checked" against local experience gained in the mining and operation of these geotechnically-matched underground facilities.

The NuMI site was investigated along the alignment using three vertical and two inclined boreholes. In addition, seismic profiling was used in areas of shallow rock cover. A conventional suite of borehole and laboratory testing was undertaken to investigate the permeability of the rock mass along the length of the boreholes.

4.3 Blocky rock behaviour

For support design, three discrete rock mass zones were identified; they were dolomitic limestone (dolostone), siltstone and shale. These rocks all had blocky structures, with natural block size determined by subvertical, orthogonal jointing and bedding. Joint spacings were wide, joint surfaces were rough and surface conditions unaltered at tunnel depth. Uniaxial compressive strength (UCS) values and rock mass classification numbers (RQD, Q, and RMR) are summarized in Table 1.

Rock support providing for mitigation against block and wedge failure was based on the use of bolts. Bolt lengths and spacing developed during design were consistent with the support recommendations of the Q, RMR and RQD classification systems for all tunnel spans. The rock mass classification systems all provided well-aligned rock support recommendations to mitigate against block and wedge failures under gravity loading.

4.4 Slaking/swelling potential

It is noted that if the classification numbers shown in Table 1 had been adopted as the sole basis for design, using any one of the three classifications, the dolomites would have been identified as requiring as much, if not more, support than the siltstones and shales. However, after reviewing the slake and swell data shown in Table 2, it was clear to the designer that the siltstone and shale would require additional support elements to counter the potential for the on-set of slake/swell behaviour. The dolostone, siltstone and shale were classified as having very high, mediumhigh and medium slake durability respectively (Deen, 1981). "Protective support" was specified in the invert (concrete "mud mat") and walls and arches

Rock unit	UCS	RQD	Q	RMR
Units	MPa	%	Index	Points
Dolostone	90	84–93	5.6-12.5	61–69
Siltstone	40	98	32.8	69
Shale	25	100	69	69

Table 1. Strength and classification data reported in the NuMI tunnels and halls contract documents.

(shotcrete) of the mined openings of the slake/swellsensitive rock units. To ensure against the onset of these types of adverse behaviour, the protective support was to be installed within a short period (typically 24 hours) of excavation.

The potential for the onset of slake/swell behavior is not considered in the RQD or RMR systems and was not brought to evidence by the Q number, despite the fact that swell potential is included in the "parameter mix" within the Stress Reduction Factor. High relative values for other constituent parameters, such as block size and joint condition, can readily act to offset a low SRF-value, attributed to the presence of swelling minerals. In such instances, the impacts of swelling ground support should be addressed separately, as the designer did for NuMI.

In general, the designer should not rely solely on classification end-numbers as they may not the support of a rock mass cannot be relied upon to predict the impacts and mitigation requirements for slake/swell behaviour. Such behaviour can have a major impact on tunnel construction and is, in fact, identified as the worst-case scenario in Terzaghi's classification.

From a practical standpoint, it is key to recognize slake/swell potential during design because, unlike other forms of instability, swell/slake degradation may not become apparent until well after mining has taken place, thus potentially creating a need for remedial action over long lengths of previously mined tunnel.

4.5 Water inflow estimate

Q and RMR classification systems discussed above include water parameters weighted according to estimates of either flow or pressure. The methodology for predicting water inflow is not specified. To estimate the NuMI tunnels' water inflow rates in the various zones, the tunnel designer opted to use the methodology developed by Heuer (1995). Heuer's empirical method was developed in large part from the backanalysis of case histories that were sited in blocky rock mass, similar to those of the NuMI tunnels. As such, it was considered an appropriate empirical method for calculation of NuMI tunnels inflows. Estimated inflows calculated following the Heuer method are shown in Table 3.

Rock unit	Slake durability	Swell potential	
Units	%	%	
Dolostone	99	N/A	
Siltstone	96	3.0	
Shale	89	2.7	

Table 2. Slake surability and swell potential data reported in the NuMI tunnels and halls contract documents.

Table 3. Water inflow predictions reported in the NuMI tunnels and halls contract documents.

Zone	Length (m)	Max steady state (gpm)	
Glacial till	30		20
Dolostone	500		170
Siltstone/shale	800		300
Vertical shafts (2)	150		110
Total	1480		600

The NuMI rock tunnels are sited at depth of up to approximately 100 m below the water table.

Following Heuer's methodology, transient flow rates within any section of tunnel were estimated to be up to twice the maximum steady state rates listed in the table. Estimated inflows were compared to local experience in mining tunnels and sinking shafts. A total inflow rate of 150 gpm was targeted for the final condition at the conclusion of the tunneling contract after grouting had been completed. Initial flow rates encountered within the tunnel were within the ranges identified in the Table.

4.6 Overstress

High horizontal bedrock stresses are known to exist in the upper mid-west region of the United States. The potential negative impact of these high stresses was mitigated during design. The rock tunnel layout and sections, as designed, were based on the use of drill and blast technologies with time and location provisions placed on the installation of ground support.

During design reference was made to the performance of other chambers in the region that were constructed at a similar depth in the same or similar rock units. These excavations were also mined using drill and blast techniques with ground support installed on a "round-by-round" basis, after mucking and scaling had been completed. Similar support provisions were placed in the NuMI contract to ensure that all drill and blast headings were fully scaled and bolted-up to within one meter of the advancing face, within eight hours of each blast. The support installation provisions were thus consistent with local practice and with the general recommendations for the support of tunnels exposed to moderate and high stress conditions, discussed by Broch and Sarheim (1984). These provisions satisfactorily mitigated against overstress failure within all the reaches of the NuMI tunnels that were mined using drill and blast techniques.

Although no overstress phenomena were observed in any of the drill and blast reaches of the NuMI tunnels, overstress failures did occur in sections of the tunnel that were mined using a TBM. The Contractor, S.A.Healy, chose to modify the design of sections of

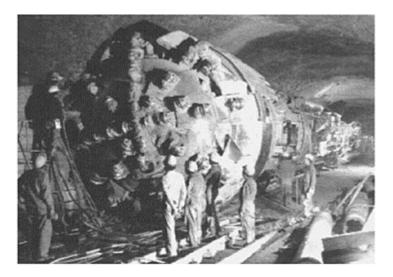


Figure 4. Tunnel boring machine system.

the NuMI tunnels to allow for the use a TBM in dolostone, siltstone and shale, as part of a value-engineering proposal.

Figure 4 shows an assembled TBM and back-up system similar to the one the contractor chose to use to excavate the NuMI alignment. The cutterhead and shield are shown in the foreground with the trailing gear laid-out behind.

When making its value engineering decision to change mining methods and means, the contractor committed to a redesign of the ground supports so as to be consistent with their preferred mining method. As can be seen from the photograph in Figure 4, the physical presence of the TBM within the tunnel heading precluded the level of access to the heading that would have been necessary for scaling and installation of the rock bolts and application of shotcrete and mud mat as laid-out in the original contract provisions. The contractor was obliged to pursue alternate support strategies that could to address the potential impacts of overstress and slake/swell behaviour on the TBM operations while still delivering an acceptable product to the owner.

In the TBM-mined sections of the siltstone and shale, overstress conditions developed above the shielded section of the TBM within a few meters of the cutter head. This behaviour was predicted from a simple consideration of the strength: stress ratio, as referenced in the Q-system and discussed by Broch (1984). The contractor had foreseen the likelihood of this behaviour and had made provision to install a wire mesh and steel channel canopy to contain and support the failed rock as it was deposited from the advancing TBM shield. Rock bolts installed from the TBM platform supported the canopy structure.

Contact grouting was performed after mining was complete to consolidate the failed zone of rock present along the crown of the tunnel. Post-excavation surveys measured the average depth of overstress failure at the centerline of the tunnel to be approximately 30 cm. The failed zone was generally limited in width to an eleven to one o'clock section of the arch.

Alternate measures to mitigate or redress the effects of swell and slake degradation on the siltstone and shale were used in order to minimize interference with the TBM operations. In the arch and sidewalls, the contractor took the risk of delaying the placement of the shotcrete and invert "mud mat" until after the completion of TBM mining. In the shale, a temporary coating of "Texflex" (Fosroc patent) was applied to the arch and sidewall surfaces.

Permanent shotcrete linings were placed after the TBM operation was complete. Cleaning and scaling was necessary in unprotected sections of the siltstone and shale sidewalls to re-establish sound surfaces prior to shotcreting. In the invert, where the contractor had opted not to place a mud-mat, a significant amount of additional excavation and concrete placement was required to remove overstress-failed and waterdeteriorated mud-rock materials.

In this instance, the TBM-bored tunnel required more ground support than the drill and blast alternative. This case underlines the fact that the engineer should not automatically assume that the TBM-bored excavation will require less support than a blasted alternative. This is another example in which the use of a classification system could have led to the selection of an inappropriate support solution.

The application of any of the number-based classification systems described above would not have resulted in an accurate prediction of rock support requirements or resulted in the recognition of other key issues that required specific mitigation plans.

4.7 Heavily fractured and soil-like zones

The "Risk Thermometer" in Figure 5 indicates that even in un-deformed (layer-cake) sandstone and limestone formations, the risks of a tunneling operation (TBM) encountering an EMA are relatively low, averaging 25 and 50 kilometers between events.

However, even where the regional geologic conditions in the area would suggest that the likelihood of such an event is low, the consequences of such an event are always large. It behooves the geologists and engineers to investigate and, as appropriate, develop mitigation plans to deal with such conditions as an integral part of the rock mass characterizations process.

During the early stages of the NuMI site investigation work, evidence suggesting the presence of a fracture zone was uncovered. The balance of the site investigation work did not allow for the existence of this fracture zone to be confirmed or denied. Given this uncertainty and the potential severity of impact of such a feature on the mining operation, provision for encountering two such features ("fracture zones") was written into the scope of the contract. Changes to

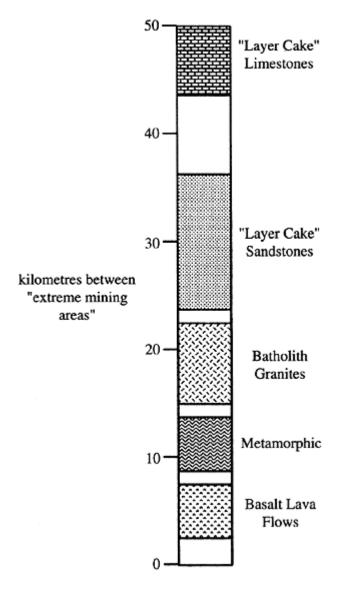


Figure 5. Risk of encountering "Extreme Mining Areas" (EMA) with various "geologic regimes"

the mining process, including ground probing, treatment, support and excavation proce dures were specified to provide for mitigation against the impacts of such features.

As it turned-out no such fracture zone was encountered during tunneling. However, it is suggested that it is a reasonable policy to expect something a little worse than has actually been uncovered by the site investigation campaign. As noted by Boden (1985) "It is most unlikely that any ground investigation for a rock tunnel will encompass the full range of rock strengths, which will be encountered during construction."

Faults and fracture zones, although relatively uncommon in most geologic settings, can have a severe impact on a project's cost and schedule.

5 CONCLUSIONS

Rock Mass Classification systems can be powerful tools that the engineer can use to develop an early appreciation of a rock mass and its strengths and weaknesses. However, the single number indices or ratings they generate may inadequately weight the risk of failure for conditions that are underrepresented in the empirical data sets from which they we derived. In particular, classifications may not sufficiently address the need for additional support elements where overstress, swelling, or soil-like behavior is encountered.

In addition, the founding data sets, dating back to the 1970s, do not adequately represent the use of newer excavation technologies, most notably TBMs and integrated steel canopy support systems. The rock mass classifications currently in use risk redundancy without a thorough update incorporating a wider range of rock mass behaviours and a full-recalibration to present day excavation practices

Designers need to guard against oversights that result from a heavy reliance on classifications alone. To avoid such oversights, it is suggested that a full suite of failure modes be considered and analyses performed to account for the impact of these missing or underrepresented factors on support needs. The case history discussed above brings into evidence some factors that need to be predicted and mitigated during the planning stage of a tunnel drive.

For all its natural complexity, rock mass failures within a tunnel are commonly associated with only a handful of different failure mechanisms. Although failures most often are associated with discontinuities, other modes of failure are also common. Their potential occurrence should be investigated during the design process, as an unexpected encounter can significantly impact the costs and schedule of a tunnel project.

Armed with adequate knowledge of both most likely and less likely but potentially severe conditions, the design team is better prepared to mitigate the serious problems that can derail an otherwise welldesigned project.

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9 Design

Rock reinforcement design for overstressed rock using three dimensional numerical modeling

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ABSTRACT: A procedure is presented for the design of reinforcement for highly stressed rock based on 3D numerical stress analysis using the MAP3D code. Modelling requires extensive characterisation of the rock mass strength and deformability and appropriate characterisation of the stress field. The numerical model is calibrated using a Rock Mass Damage Criterion and a Rock Mass Failure Criterion that are calibrated to observations of in situ cracking. These criteria define an outer damaged or cracked zone and an inner, failed or broken zone. Examples are used to show how the extent and dimensions of these zones can be determined by post-processing the modelling results and how the boundary of these zones can be used to dimension the primary reinforcement scheme.

1 INTRODUCTION

The effectiveness of a particular reinforcement scheme in terms of its density and length can be assessed using empirical strategies, theoretical methods and geotechnical instrumentation (Windsor and Thompson, 1993). In addition, detailed stability analysis may now also include the calibration of a numerical model program to simulate the excavation sequences in order to assess the role of stress change on the rock mass environment. Three dimensional numerical modelling results can then be used to determine the overall stability around underground excavations, where zones of damage, or failure can be estimated and then used to determine the required length and capacity of a reinforcement scheme. The required input parameters consist of the in-situ stress profile with depth, the strength and deformational properties of the rock mass as well as the excavation steps and their sequence.

2 ROCK MASS STRENGTH & DEFORMABILITY

The rock mass compressive strength is a measure of the peak load carrying capacity of a rock mass. It is defined as a proportion of the intact rock strength due to the presence of geological discontinuities (Hoek and Brown, 1980). The rock mass strength is defined here as the limiting load required for stress driven failures to initiate and propagate around underground excavations. In modern engineering mine design, the rock mass strength is usually estimated prior to excavation using borehole data (i.e. as part of the orebody delineation process, when full core is available) to determine the variability of the intact rock properties and the rock mass classification parameters throughout the orebody. The intact rock parameters (uniaxial compressive strength and the elastic constants E and ν) can be determined from a number of representative holes taken from the full set of exploration holes (Figure 1)

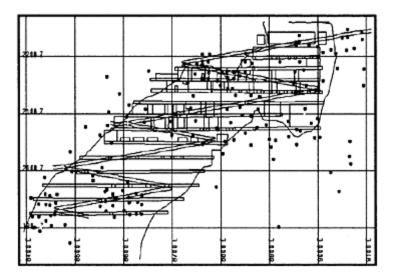


Figure 1. Typical distribution of exploration holes in an orebody.

thus allowing the characterization of the entire orebody.

Usually, the uniaxial compressive strength and its variation for each rock type present can be defined such as in Table 1. Alternatively it may be presented using modelled contours across a particular unit such as the strength distribution in the hangingwall boundary as shown in Figure 2.

The actual value of rock mass strength and deformability depends upon the geometrical nature and strength of the geological discontinuities, which can be estimated by using empirical methods that rely on rock mass classifications. Table 2 presents some typical average results for the same rock units described before in Table1.

Rock type	Sample number	UCS (MPa)	STDev (MPa)
Hangingwall rock	9	114	25.1
Orebody	11	107	42.2
Footwall rock A	7	139	40.1
Footwall Rock B	15	107	23.0

Table 1. Average uniaxial compressive strength per rock type.

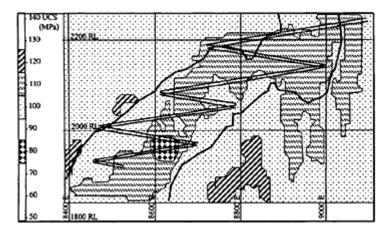


Figure 2. Modelled UCS variability across an orebody hangingwall boundary.

Table 2. Average rock mass properties per rock type.

Rock mass type	E_m (GPa)	σ_{cm} (MPa)	φ _m (°)
Hangingwall rock	32	46	40
Orebody	31	44	44
Footwall rock A	32	57	40
Footwall rock B	31	44	44

Where: E_m is the deformation modulus, σ_{cm} is the uniaxial compressive strengths, and Φ_m is the friction angle of the rock mass.

3 IN SITU STRESS

Reliable evaluation of *in situ* stress is an important phase in the analysis and design of underground excavations, particularly when evaluating excavation stability with the aim of preventing stope/pillar wall failures. Consequently, over the last seventy years considerable effort has been invested by numerous research organizations in finding suitable methods to quantify Earth's crustal stresses. In cases where excavation access is available, the overcoring method using the CSIRO HI cell has proven to be an accurate and reliable method of measuring the complete 3D stress tensor. In addition, in the last few years the Western Australian School of Mines (WASM) has developed a technique to determine the complete 3D stress tensor using the Acoustic Emission method (Villaescusa et al, 2003). The practical advantages of the WASM AE technique revolve around the fact that the state of stress may be quantified for any point where oriented exploration core can be obtained. This negates the previous restriction for the existence of an excavation from which to conduct the measurements.

The results shown in Figure 3 compare the stress tensors obtained by the CSIRO HI cell and the WASM AE method for the orebody shown in Figures 1 and 2. The comparison involves a two point WASM AE stress profile defined over a 100 m interval and a single CSIRO HI overcoring result at a shallower depth separated by 150 m. The principal stress magnitudedepth relationships and the principal stress orientations for the two point WASM AE profile and the single CSIRO HI result are given together in Figure 3.

The data shows that there is excellent agreement between the extrapolation of WASM AE magnitudes of the principal stresses compared with those obtained by the CSIRO HI cell overcoring and between the principal stress orientations indicated by overcoring at 363 m depth compared to that obtained by WASM AE at 493 m depth. Comparison of the two WASM AE measurements at 493 m and 595 m indicate a small rotation that effectively 'flips' the major and intermediate principal directions to diametrically opposite positions on the projection. The relative variation for each principal stress magnitude is shown in parenthesis under each projection of WASM AE principal orientations.

4 NUMERICAL MODELLING

In most cases of underground mining, the induced stresses may be determined using linear elastic numerical modelling. The required inputs are the in-situ stress field with depth, the estimated deformational properties of the rock mass and the chosen extraction sequence. In this study the elastic version of the

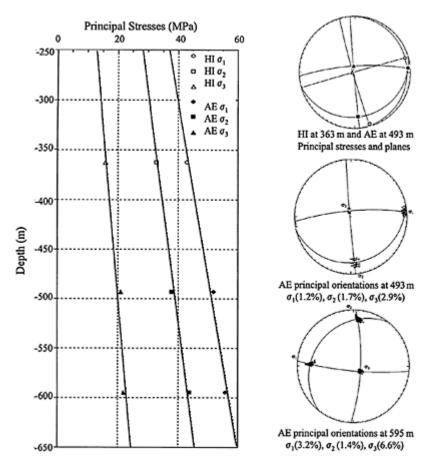


Figure 3. Comparison of a three point AE stress measurement profile with a single point CSIRO HI Cell stress measurement.

computer program Map3D was used to determine the stress distribution around the underground excavations. It must be understood that the results are used in conjunction with structural information (for example large fault behaviour) in order to interpret any excavation option analyzed as well as their respective reinforcement strategies. Typical output from numerical modelling includes stresses and displacements. These can then be compared with empirical failure criterion established for the different domains around the underground excavations within an orebody. It must be emphasized that any predictive models must be calibrated (validated) against field data and observations using either visual methods and/or geotechnical instruments.

Although linear elastic modelling can be used to estimate the level of damage and the extent of the failure zones, non-linear models are required to simulate any resultant stress

re-distribution from such failures. Progressive orebody extraction may induce several phases of post-peak behaviour in a rock mass and similarly, small changes to the stress field induced by distant extractions may induce significant rock mass damage around a particular excavation wall.

5 FAILURE CRITERION

Experience through correlation of underground observations and geotechnical instrumentation with numerical modelling results suggest that a rock mass is damaged when a range of induced stress levels exceeds a certain site dependent threshold as shown in Figure 4. Below the damage threshold the response is elastic and usually very little damage can be observed. However, with increased overstressing increased damage is experienced. The actual damage level reached depends upon the amount of overstressing and beyond the initial damage threshold a zone of potential overbreak (POB) is reached. Increased stress beyond this level may cause stress driven failures and eventually the rock mass may become unsupportable.

Consequently, a rock mass is neither strictly failed nor unfailed, but rather for similar confinements, there is a range of stress levels where increasing excavation damage is experienced. A Rock Mass Damage Criterion can be defined as follows:

 $\sigma_1 = A + p \sigma_3$

(1)

where A and p are site dependent constants. Back analysis of numerical modelling over a number of years (Wiles 1998, 2004) suggests that p normally takes on a value near unity.

Figure 5 shows an empirical damage criterion established from back analysis of cavity monitoring system (CMS) surveys for a primary stope located at a deep underground operation in Western Australia.

The criterion expressed by Equation 1 is also conceptually represented by the lower line in Figure 6 and can be interpreted to represent the stress level where seismicity is observed to occur. In addition, borehole cameras can be used to directly observe the amount of damage in the form of increased fracture frequency.

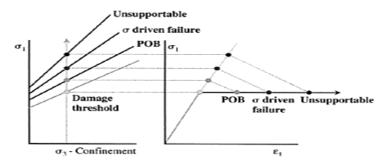


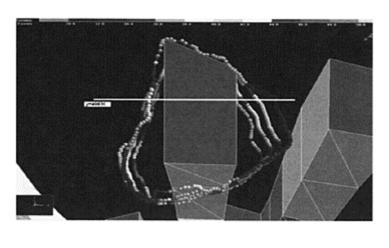
Figure 4. Different levels of stress driven damage and failure.

Geotechnical instrumentation shows that when this stress level is exceeded a loss of rock mass cohesion is experienced. However, a considerable degree of residual frictional strength (i.e. interlocking) is still available. Nevertheless, the rock mass is visibly cracked and may unravel and disintegrate if it is not held together by a ground support scheme.

Another criterion that can be readily identified (upper line in Figure 6) is commonly called the Rock Mass Strength Criterion and is defined as follows:

 $\sigma_1 = B + q \sigma_3$

(2)



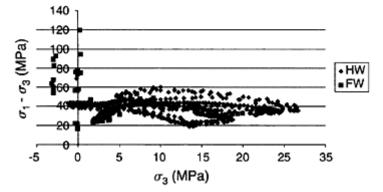


Figure 5. CMS profile of failure and damage criterion from back analysis using Map3D.

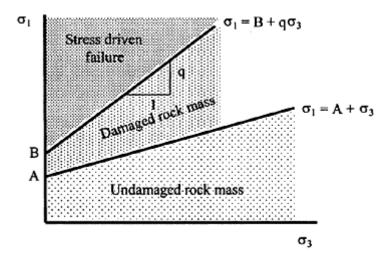


Figure 6. Rock mass damage zones.

where B is the Uniaxial Compressive Strength (σ_{cm}) of the rock mass and q is related to the rock mass friction angle (Φ_m) by tan²(45+ $\Phi_m/2$).

Strength Factor A =
$$\frac{\sigma_{cm} + \sigma_3 \tan^2(45 + \phi_m/2)}{\sigma_1}$$

This criterion represents a stress level at which failures can be considered to be stress driven. When the stresses reach this level the interlocking is overcome and the rock mass undergoes considerable non-linear deformation. This deformation is driven by large forces that may not be held back by ground support schemes. In fact, ground support must be able to move with this deformation if the failed material is to be contained.

Data from a number of mines exhibiting brittle rock response suggests that A and B have similar magnitudes, and the two criterion may meet at $\sigma_3=0$. It is anticipated that this may not be true for more compliant rock types. In addition, the rock located within the zone defined between the two criteria can be

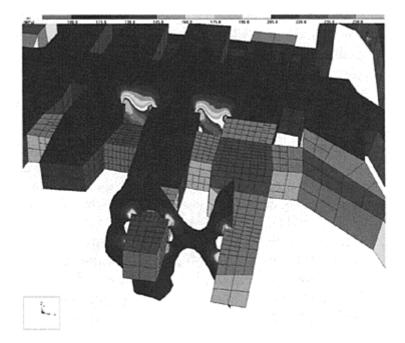


Figure 7. Modelled induced stresses in pillars using Map3D.

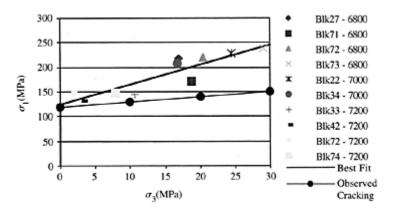


Figure 8. Back analysis of pillar failures.

considered to be damaged. As overstressing increases from the lower criterion to the upper one, the rock mass becomes progressively more sensitive, in that it is easier to trigger an unravelling failure, for example by blasting nearby.

While the rock mass strength failure criterion discussed above can be estimated by using empirical methods that rely on rock mass classification, correlation of underground observations and geotechnical instrumentation with back-analyses is used to verify whether the estimate is correct and refine the actual values.

For example in Figure 7, the two pillars towards the back were observed to fail right through to the core, while the pillar in the foreground experienced side wall spalling only Elastic modelling can be used to determine the stress levels respectively in the core and side walls. These stresses can then used to verify the failure criterion. By repeating this type of back-analysis for many observations in situ, the site specific rock mass compressive strength representing stress driven failure can be determined as shown in Figure 8.

Also shown in Figure 8 are results from a back analysis of locations where cracking was observed in boreholes. This provides values to be used in the failure criterion described above (Equations 1 and 2):

$\sigma_1 - \sigma_3 = 120$	Rock Mass Damage Criterion
$\sigma_1 = 124 + 4.1 \sigma_3$	Rock Mass Failure Criterion

6 ROCK REINFORCEMENT DESIGN

Whether the strength parameters defined in Equations 1 and 2 are determined from back analysis, or estimated using empirical methods that rely on rock mass classifications, the design of the rock reinforcement in overstressed rock can be achieved using three dimensional numerical modeling. This can be achieved by assuming that the ground response can be described by two categories: broken ground and cracked ground as shown in Figure 9.

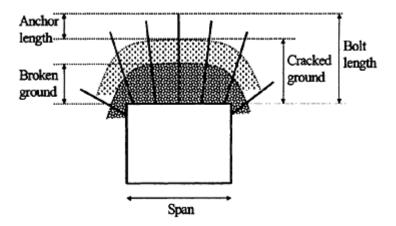


Figure 9. Zones used for rock reinforcement design.

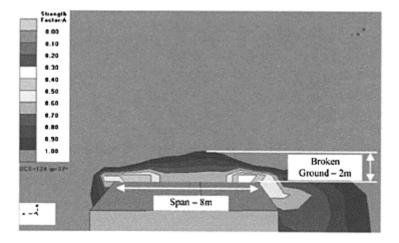


Figure 10. Contours of $(124+4.1\sigma_3)/\sigma_1$.

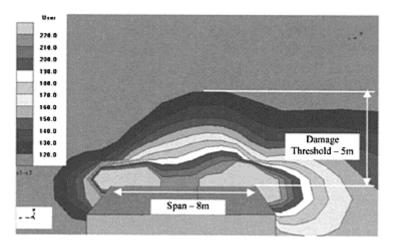


Figure 11. Contours of $(\sigma_1 - \sigma_3)$.

The broken ground is ground that has undergone stress driven failure and represents the dead weight that our support needs to suspend. This will be determined using the rock mass stress failure criterion. Consider a highly stressed location shown in Figure 10. To determine the depth of broken ground, the values of strength divided by stress have been contoured, or $(124+ 4.1\sigma_3)/\sigma_1$. The results show that the broken ground depth extends 2 metres into the back.

The cracked ground defines where the reinforcement anchoring begins. This will be determined using the rock mass damage threshold criterion defined earlier. Consequently, the depth to the damage threshold is determined using the contoured values of $(\sigma_1 - \sigma_3)$ as shown in Figure 11.

In this location the dimensions and extent of the cracked or damaged zone (and within this zone the dimensions and extent of the broken or fractured zone) have been determined. The results suggest that cable bolts are required across the 8 m span and will need to extend past the 5 m deep damaged zone to be anchored within intact rock. The density of cable bolts may. determined by considering the mass of the damaged zone across the span of the opening. The type, stiffness and installation timing of the cable bolts chosen will depend on the expected velocity of loading, both for the current circumstance and for future mining induced stress changes. Investigation of the 2 m deep broken or failed ground during the back analysis stage will indicate if and what type of rock bolts and mesh are required to retain the broken ground between the cable bolt array spans.

At other locations stress levels may be insufficient to induce stress driven failures. Therefore, at such locations this method would predict zero depth of broken ground and alternative failure mechanisms and alternative analysis methods depending on the structural geology and geometry of the openings must be considered. In medium to low stress conditions there are basically three cases of rock mass to consider: massive, stratified, and jointed rock. In massive rock a simple two dimensional, elastic analysis of the opening and the stress field may predict mild spalling as opposed to deep fracturing. In stratified rock, beam or plate theory may predict shearing and dilation to occur depending on the orientation of the stratigraphy and the orientation and shape of the opening. This may lead to bending and buckling with step path failures through the layers or cantilever action and guttering of the layers, which is common in coal mining collapse mechanisms. In jointed rock, block theory may be used to the stability of blocks of rock that may translate or rotate towards the opening. This mechanism may initiate with the loss of individual blocks but may propagate to a progressive collapse of the block assembly around the opening. In each case it will be necessary to predict the dimensions and extent of the failure zone and provide a reinforcement and or support scheme suitable for both global and local stability. The analysis methods and procedures for reinforcement design in these circumstances have been given by Hoek and Brown (1981) and Brady and Brown (1985).

Once an initial design has been formulated this modeling method may also be used to evaluate how other excavation stabilization techniques affect the rock reinforcement requirements. This would include modelling of alternative sequences, reduced spans and using backfill.

7 CONCLUSIONS

A procedure has been given for the design of reinforcement for highly stressed rock based on 3D numerical modelling using the Map3D code. The procedure involves a sequence:

- 1. Characterisation of the rock mass strength and deformability.
- 2. Characterisation of the stress field.
- 3. Definition of the mine geometry and excavation sequence.
- 4. Modelling of the stress redistribution due to excavation.
- 5. Back analysis to determine:

a. A Rock Mass Damage Criterion

- b. A Rock Mass Failure Criterion
- 6. Post-processing of stored analysis results to define the outer, damaged or cracked zone and the inner, failed or broken zone.
- 7. Primary reinforcement is dimensioned on the geometry and mass of the damaged and failed zones.
- 8. Secondary reinforcement and or support is dimensioned on the geometry and likely behaviour of the failed zone local to the excavation surface.

This procedure is suitable for hard rock mines which have obtained sufficient data to properly characterise the rock mass and the stress field. The back analysis component is required in all cases in order to calibrate numerical model predictions and the damage and failure criteria to in situ observations of cracking. Closing the analysis with observations in this manner ensures a progression to appropriately dimensioned primary reinforcement.

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Issues in selection and design of ore pass support

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ABSTRACT: This paper provides a critical review of issues associated with ore pass support. A successful ore pass design has to ensure the stability of the excavation while at the same time allowing for the uninterrupted material flow through the ore or waste pass system. The success of reinforcement and support systems has often been questionable.

1 INTRODUCTION

1.1 Background

The design of support systems for underground excavations has been addressed in several publications including Hadjigeorgiou & Charette (2001), Hoek et al. (1995) and others. The design of support for ore pass systems has not however received the same level of attention as accorded to other mining infrastructure. This is unfortunate given the importance of ore pass systems on economic transport of material in underground mines. An ore or waste pass is a vertical, or steeply inclined, excavation that allows for the gravity transport of broken ore and waste from one level to another, Figure 1.

The purpose of ground support in ore passes is twofold: to ensure the stability during the excavation process and to prevent, or mitigate, the effects of wall failure during the useful life of a pass.

2 ORE PASS STABILITY

2.1 Traditional stability considerations

The stability of underground excavations is controlled by stress and structure. This has been addressed conceptually by Hoek et al. (1995) and Martin et al. (1999) who linked rock mass quality as represented by RMR to the ratio of the maximum far field stress (σ_1) to the unconfined compressive strength (σ_c).

The influence of excavation size with respect to the relative stability has been addressed by Hudson (1989). In a jointed rock mass there is a convincing case that the stability of the excavation decreases with

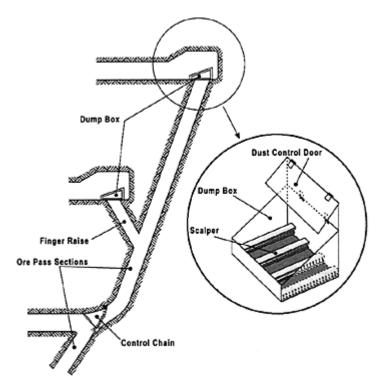


Figure 1. A typical ore pass system.

increase of the ratio of excavation dimension/mean discontinuity spacing. Usual ore pass dimensions range from 1.5 to 4 m which is equivalent to small tunnels.

The design of vertical or inclined excavations can be more complex than horizontal excavations in tabular ore bodies when ore pass systems traverse several geological regimes.

Ore passes are most often excavated by drill and blast techniques or raise bored. The method of excavation can have an adverse influence on the stability of an ore pass.

Mechanical excavation results in less ground disturbance thus mitigating the risk of instability. The stability of raise bored shafts has been investigated by McCracken and Stacey (1989). Using the Q system as an indicator of rock mass quality, they introduced a series of modifications to facilitate the design of vertical and/or inclined excavations. The modifications can account for wall stability as the controlling factor (rather than roof), excavation orientation relative to structural features and rock weatherability. It should be noted, however, that most raise bored excavations do not use reinforcement.

2.2 Influence of material flow

Material flow in ore pass systems can have an adverse effect on the stability of excavations. This can be due to direct abrasion of exposed surfaces or through the impact of material on the walls. A further complication arises if ore pass systems hang-up. Under these circumstances it is not unusual for the operators to restore flow by blasting in the ore pass. This practice can result in damage to the walls and can further endanger the integrity of the ore pass.

3 ORE PASS SUPPORT

3.1 Background

The design and installation of ore pass support is influenced by several factors including:

- ground conditions
- in situ stress regime
- mining induced stresses
- method of excavation
- excavation dimensions
- vicinity of other infrastructure
- required capacity for material transfer
- access to install support
- design life
- consequences of failure

It is often convenient to differentiate between support installed during development and support necessitated due to instability, i.e. rehabilitation. Ore pass rehabilitation is expensive and, more often than not, unsuccessful. For practical purposes it is expedient to treat separately support installed within the ore pass and support installed from access points. As for other excavations the time of installation of support is critical if wall degradation is to be prevented.

3.2 External ore pass support

Wall sloughing can result in the introduction of large blocks in the ore pass. There is ample empirical and theoretical evidence that large blocks, relative to the

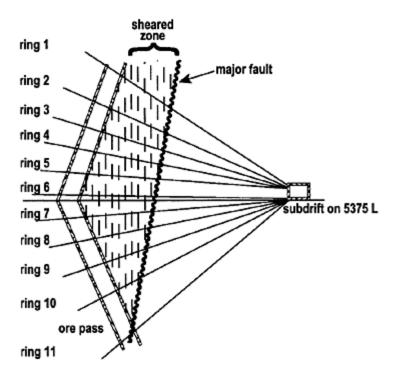


Figure 2. Layout for reinforcement rings for sheared zone, after Singh (1973).

ore pass cross section, result in interlocking arches that inhibit material flow. Quite often the only release mechanism under these circumstances is the use of explosives. This is a time consuming, expensive process that has the undesirable consequence of damaging the walls.

An interesting case study has been presented by Singh (1973). Excessive wall sloughing required recurring blasting within the ore pass to restore flow. Following field investigations it was decided to reinforce fractured zones around the existing ore passes, Figure 2. The mine implemented a comprehensive support system of grouted cable bolts, rebar, pre-stressed grouted rock bolts and shotcrete.

The installation of this support system resulted in a significant decrease in the number of blasts to release hang-ups at the chute area. This reinforcement strategy was cost effective in direct savings and in ensuring no interruption to mine production.

Clegg & Hanson (1992) report on the rehabilitation of the #1 ore pass at Fraser mine. This ore pass was shut down in early 1990 after less than a year of operation (about 300000 tons of throughput). An investigation revealed significant damage on the hanging wall and the walls of the pass. An adjacent drift was used to install single cable bolts. In order to reach the heavily damaged portion above the level itself, an access raise and drilling crosscuts had to be excavated. Birdcage cables were used in the upper part of the pass. Swan (2004) reports that the rehabilitation was successful and the ore pass remains in operation to this day. It has been argued that the success of this reinforcement strategy was due to the fact

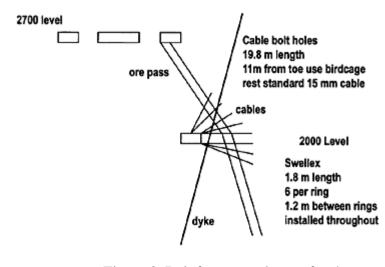


Figure 3. Reinforcement layout for the Lockerby ore pass, after Clegg & Hanson (1992).

that this particular ore pass was not exposed to seismic activity.

The same authors document the reinforcement of an ore pass extension in a seismically active zone of Lockerby mine, Figure 3. Pre-support birdcage and standard cables were installed from a drift adjacent to the ore pass as a means of stabilizing a seismically active poor quality rock zone (dyke contact). Additional support inside the pass was provided by 1.8 m coated Swellex bolts on a staggered 1.2×1.2 m pattern. Soon after rehabilitation, the ore pass experienced seismic activity, O'Hearn (2004). This resulted in wall degradation with the ore pass opening up to 14 m of within 8 months from the time of rehabilitation. Wall instability resulted in large blocks of rock in the ore pass and hang-ups. Efforts to restore flow were unsuccessful and the mine had to resort to using trucks to transfer ore from one level to the other.

An analysis of the microseismic data, geological mapping and 2d distinct element models at Lockerby suggested that the seismic events at the ore pass were of the "fault-slip" type, Canadian Rockburst Research Program (1995).

3.3 Internal ore pass reinforcement

There is an absence of generally accepted guidelines for the selection of reinforcement specific to ore passes. The only specific guidelines for ore pass reinforcement are attributed to Hambley (1987). It was suggested that RQD could be used to select reinforcement and support guidelines, Table 1.

Hambley (1987) does not comment on the choice of bolts. Nevertheless, he advocated the use of rules by Alexander and Hosking (1971) to provide guidelines for bolt length and spacing.

 $L \ge 6.0 + 0.004 W^2$

(1)

(2)

RQD	Rock bolts	Shotcrete	Cast concrete
Excellent (RQD>90)	Spot.	None	No
Good (75 <rqd<90)< td=""><td>Pattern bolting. Wide spacing.</td><td>Local</td><td>No</td></rqd<90)<>	Pattern bolting. Wide spacing.	Local	No
Fair (50 <rqd<75)< td=""><td>Pattern bolting. Medium spacing.</td><td>4 in. (or more as required)</td><td>No</td></rqd<75)<>	Pattern bolting. Medium spacing.	4 in. (or more as required)	No
Poor (25 <rqd<50)< td=""><td>Pattern bolting. Close spacing.</td><td>6 in. (or more as required)</td><td>Perhaps</td></rqd<50)<>	Pattern bolting. Close spacing.	6 in. (or more as required)	Perhaps
Very poor (RQD<25)	Pattern bolting with mesh. Very close spacing.	6 in. (or more as required) followed by cast concrete liners	Yes

Table 1. Support guidelines for ore passes, after Hambley (1987).

Excellent rock Spot bolting

Good Rock $\frac{1}{2}L < S < \frac{3}{4}L$	(3)
Fair rock $\frac{3}{8}L < S < \frac{1}{2}L$	(4)
Poor rock $\frac{1}{4}L < S < \frac{3}{8}L$	(5)
Very poor rock $S \leq \frac{1}{4}L$	(6)

where *L*=bolt length (ft); *W*=span of excavation (ft); *S*=spacing (ft).

Although this may be the only guidelines specific to ore passes, they do have some major shortcomings. In the first place the use of horizontal span, although suitable for drifts is questionable for ore pass systems. If these guidelines are applied to typical ore pass configurations, i.e. openings from 1.5 m to 4 m, the use of Alexander and Hosking (1971) guidelines would result in bolt lengths of 1.85 to 2.00 m. This would suggest that there is little sensitivity for the range of values. In practice the maximum bolt length is limited by the limited space for its installation.

Applying these spacing guidelines would result in a range of reinforcement patterns. A review of data from Quebec mines has revealed that irrespective of the merits or shortcomings of these recommendations they have little semblance to what was actually observed in the mines.

In fact, in the Quebec database, all bolts installed in ore passes were between 1.0 m to 1.8 m long. The bolts were in patterns that varied from 1.0 m by 1.0 m to 1.2 m by 1.2 m. A tighter spacing $(0.6 \text{ m} \times 0.6 \text{ m})$ was used in three sections of an ore pass. There was no real clear trend between the level of reinforcement and the observed degradation (ore pass enlargement).

At Premier Mine in South Africa, rock bolts and cable bolts were found ineffective as a means of ore pass reinforcement, Bartlett et al. (1992). In fact there was no observed correlation between reinforcement strategy and degradation in the ore passes. This would imply that other factors come into play. At Premier, installation of concrete lining in production ore passes was found effective although expensive and time consuming to install. Liner wear was also noted as a potential problem.

3.4 Choice of rock reinforcement

In Quebec mines the majority of vertical or inclined excavations are constructed using Alimak. This facilitates the installation of reinforcement and support. Lessard and Hadjigeorgiou (2003) have provided a comprehensive review of ore pass engineering practice in 10 underground mines in Quebec. Ore passes excavated using raise boring or drop raising techniques did not use reinforcement.

Resin-grouted rebar constitute the most popular reinforcement for ore pass systems. Nevertheless, the most recently developed excavations are reinforced by resin grouted short cable bolts. It should be noted that cement grouted cables were used in conjunction with resin grouted rebar. Only one Quebec mine used fiberglass rebar for selected sections. In this operation their installation proved difficult. Furthermore, as the installed fiberglass rebar did not control wall degradation the mine did not use them further.

At Creighton Mine in Sudbury, Oliver et al. (1987) argued that grouted fiberglass dowels provide the best reinforcement in ore passes. In their experience rebar bolts are not as efficient as they tend to vibrate when struck by passing muck. This results in a break down of the grout bond.

Clegg & Hanson (1992) rely on experience from Falconbridge mines to suggest a reinforcement strategy for ore passes. For low quality rock, defined by the presence of small in situ block size, they recommend stiff support such as pre-installed bird cage cables. In ore passes that traverse good quality rock (large block size) they recommend the use of coated Swellex as they are more resistant to dynamic loads.

The influence of jointing in the stability of ore passes has been addressed by Stacey & Bartlett (1990). Based on structural data in blocky rock, they generated a series of charts linking the probability of occurrence of unstable blocks in the ore pass walls and selected reinforcement patterns. The proposed reinforcement was grouted wire rope or flexible cable bolts.

More recently several Canadian Mines have used resin grouted short cable bolts, for example the Strand-Lok Cable bolt system, as it can dissipate impact shock. The preference for cable bolts over rigid rock bolt reinforcement has also been advocated by Hagan & Acheampong (1999).

3.5 Influence of material level in an ore pass

Several mine operators aim to keep an ore pass full as this practice mitigates the results of impact loads on the side walls. A further perceived advantage of keeping an ore pass full is that it can provide a confinement that contributes to the stability of an ore pass, Kazakidis and Morrison (1994).

Keeping an ore pass full requires strict procedures which may not be easy to implement and enforce while respecting production constraints. Furthermore, certain types of ore, when left stagnant are susceptible to hang-ups.

4 LINERS

4.1 Case for liners

The case for liners in ore pass systems has been made by Hadjigeorgiou & Lessard (2003). It has been reported that liners had some success in:

- Improving the structural stability of an ore pass.
- Resisting uncontrolled enlargement of an ore pass volume due to wear, impact of rock fragments, etc.
- Improving material flow.

Furthermore, it is recognized that the use of liners can help to protect installed reinforcement in the ore pass.

4.2 Liner applicability

It is accepted that liners can prolong the longevity of an ore pass, Stacey and Swart (1997), Brummer (1998) and O'Hearn and Somers (2003). Stacey and Swart (1997) recommend liners for ore passes in weak rock or in fissile, scaling or closely jointed blocky rock. Under these conditions liners can control or mitigate enlargement of ore pass dimensions. Brummer (1998) linked ore pass expected lifetime with the stress state (σ_{max}) and the uniaxial compressive strength of intact rock (σ_c). Based on data from operations under high stress conditions, he concluded that there is a direct relationship between total rock passed and the ratio (σ_{max}/σ_c). O'Hearn and Somers (2003) reiterate a widely held view that a main advantage of cementitious liners is to protect ore pass reinforcement.

Shotcrete liners were recommended by Hambley (1987) for RQD<75, see Table 1. Furthermore, it was suggested that an additional liner of cast concrete should be installed for RQD<25. Based on observations across Canada, these recommendations are not used.

4.3 Performance of reinforcement and support

The relationship between rock mass and ore pass operational failure was addressed for 10 Quebec mines, Hadjigeorgiou & Lessard (2003). Cavity monitoring surveys, volume reconciliation from actual tonnage

Q rating	Number of sections	-	Fotal	Failed
Non supported				
>5 (fair)		47	4	0
<5 (poor)		53	3	3
Total		100	7	3
Reinforced				
>5 (fair)		47	43	0
<5 (poor)		53	50	30
Total		100	93	30
Lined & Reinfor	ced			
>5 (fair)		47	0	0
<5 (poor)		53	19	16
Total		100	19	16

Table 2. Performance of ore pass support in Quebec mines.

capacity and/or comments from mine operators were used to quantify the expansion of an ore pass. An ore pass section is considered to have "failed" if it had expanded to twice its initial volume as recorded in the original layout. The Q system, Barton et al. (1974) was used to quantify rock mass quality. The results of are summarized in Table 2.

Referring to Table 2, there is no incidence of uncontrolled ore pass failure in any rock pass section that had a Q value greater than 5. It is suggested that liners should be considered for ground conditions where Q is less than 5.

4.4 Quantifying ore pass support costs

It follows that the choice of reinforcement and support is influenced by costs. These are often difficult to quantify as there can be particular conditions that may distort the true costs. O'Hearn and Somers (2003) provide comparative costs for different liner options for an Alimak driven ore pass 100 m long and 2.4 m×2.4 m in cross section, Table 3. Portland refers to Portland based mix design with hard aggregate, while Portland/SF is Portland cement with 8% silica fume mix design with hard aggregate. Fondag is a calcium aluminate cement (CAC) with synthetic aggregate (Alag) made from fused CAC. Fonducrete is CAC with hard natural aggregate and Alag. All mixes that are compared in

this analysis include fiber. This analysis assumed that a 150–200 mm thick liner is applied over the full length of the ore pass.

It can be seen that the use of liners results in cost increases ranging from 23% for use of Portland Cement, to 73% for use of Fondag.

	nero.				
	Cost (\$CDN)	Portland	Portland/SF	Fondag [®]	Fonducrete®
Set-up & Tear down	70,000				
Excavate	250,000				
Bolts & screen	39,000				
Liner Application		119,000	119,000	119,000	119,000
Liner Material		89,000	112,200	540,000	338,000
Rock Breaker Grizzly	300,000				
Chute Chains	250,000				
Base cost	909,000				
With liner		1,117,000	1,140,200	1,568,000	1,366,000

Table 3. Ore pass costs for different cement based liners.

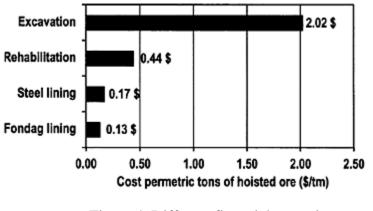


Figure 4. Different financial scenarios for ore pass development, after Cossette and Durham (2000).

The financial justification for the use of liners is highlighted if one accounts for lost revenues due to ore pass related loss of production. If for example one assumes a 4 500 tpd copper mine, a 4.5% grade and a copper price of \$1.50 CDN/lbs, revenue loss for a single day will be \$669 654 CDN.

Of interest is the analysis by Cossette and Durham (2000) on four scenarios for the development of an ore pass system, Figure 4. All costs were amortised over a ten year

period. Under consideration was: lining the existing ore pass with Fondag; using steel liners; planning for rehabilitation work after 3 years; driving a new ore pass after 4 years of operation. The most expensive scenario would have been driving a new ore pass. This would have necessitated moving permanent mine infrastructures (rock breaker and conveyor). Deciding to go with the rehabilitation option would have resulted in production losses. Under these conditions the use of Fondag lining was the most cost effective option for this operation.

5 CONCLUSIONS

This paper addresses some of the technical issues associated with the use of reinforcement and support for ore pass systems. There is empirical evidence that the use of reinforcement in an ore pass is not successful for difficult rock conditions. Site visits suggest that choice of support is often dictated by time and cost constraints and not by observed ground conditions.

It is proposed that the real costs of reinforcement and support can only be addressed by investigating the monetary consequences of failure and rehabilitation.

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Ground support—predicting when to change the pattern

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ABSTRACT: All underground mines in Western Australia must provide standard support patterns to be used for different expected ground conditions as part of their Ground Control Management Plan. The decision on when to change patterns is often masked by the prevailing ground conditions, particularly in talc chlorite ultramafic rock. This paper examines the use of Tectogenesis to predict the onset of heavily sheared ground, and hence provide a quantifiable approach for anticipating changed ground conditions. The technique was trialled at St Barbara Mines Limited (SBML) Gibraltar Underground where it showed initial success.

1 INTRODUCTION

The training of operators and operational staff in mines without daily geotechnical personnel support to recognise changed ground conditions, and the impact of these changes, has been successful. Such courses have greatly improved communications. The emphasis of this training is not on identifying the "geotechnical parameters" which are often full of unfamiliar jargon (e.g. faults, shears, joints, GSI, RQD) but on the practical observations and their impact on support requirements. It is our experience that following such courses, the level of discussion and understanding is greatly improved.

Pattern support is often specified for operations. This simplifies decision making, ensures adequate supplies, and can be supplemented if not deemed sufficient by site personnel. The decision on when to use the different hierarchies of pattern support (e.g. as shown in Figure 1) is frequently made without additional geotechnical input.

The proposed patterns and the observations necessary to select suitable ground support are appropriate and are not in question. However, at Gibraltar the conditions as to when to adopt the change in support requirements were often not immediately obvious even to trained geotechnical personnel. The purpose of this paper is to present a methodology that was trialled at St Barbara Mines with initial success. It outlines the reasons for necessitating the approach and how it was developed.

2 BACKGROUND

2.1 Gibraltar

The Gibraltar ore zone is hosted as an envelope within a talc chlorite schist which dips at approximately 60° to the east, and is generally considered conformable with foliation. At the feasibility stage the major defect sets identified were:

	5
Set	Dip/Direction
А	50° to 60° to the East
В	60° to 80° towards 240°
С	30° to 50° towards 045°
Foliation D	70° to 80° toward 090°

Regions of poor ground were anticipated to be associated with shear zones. There are two sets missing from this interpretation which subsequently became the most significant.

Set	Dip/Direction
Е	70° towards 080°
F	55° towards 290°

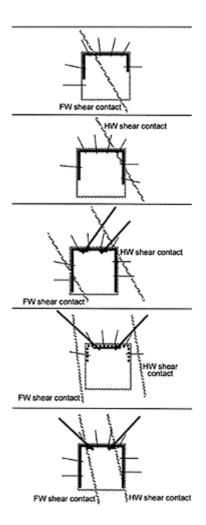
Set E is difficult to identify in a "statistical" interpretation of the data because it is subparallel to foliation. Features making up Set F were observed in the pit. They were widely spaced and not considered important at the drive or stope scale. Subsequent experience demonstrated how important these features became in determining the location of poor ground. When features from Set E and Set F intersected, heavily sheared ground was encountered.

The general layout of the economic mineralised zone is shown in long section in Figure 2.

During the initial decline development the first significant shear zone which affected production was encountered on the mafic/ultramafic contact. This occurred 20 m beyond the deepest hole drilled

for the assessment of the portal and initial decline development.

The next intersected shear was oriented at 70° towards 078° at 410 mRL. The location of this shear, when projected was anticipated to intersect the decline at 380 mRL, 360 mRL, 340 mRL and 320 mRL. This was tested with a specifically drilled diamond hole which confirmed the continuity and orientation of the feature.



Recommended support practice

Footwall shear contact is exposed in the back of ore drive. The hangingwall shear contact is >1.5m from eastern sidewall.

- Implementation of ore drive support pattern (2).
- Shotcrete and additional bolting to prevent undercutting of the western sidewall

Hangingwall shear contact is exposed in ore drive. The footwall shear contact is >1.5m from the western sidewall.

- Implementation of ore drive support pattern (2).
- Shotcrete and additional bolting to extend down to suitably reinforce contact from toppling failure.

Footwall and hangingwall shear contacts are exposed (or are likely to be exposed) in the excavation. The dip of the contacts is <65*

- Implementation of ore drive support system (2).
- Shotcrete and bolting to effectively support all sidewall brows.
- Support upgrade (b) with cable faceplates (with shotcrete) (straps if mesh surface support).

Footwall and hanging wall shear contacts are < 1.5m into the sidewall of the excavation, but not exposed (or only in lower sidewall). Dip of mineralised zone and contacts is >65°

- Implementation of ore drive support system (1).
- mesh and bolting to effectively support all sidewall brows.
- Support upgrade (a) with strap.

Footwall and hangingwall shear contacts are exposed (or likely to be exposed) in the excavation. The dip of the mineralised zone and contacts is >65*

- Implementation of ore drive support system (2).
- Shotcrete and bolting to effectively support all sidewall brows.
- Support upgrade (a) with cable faceplates (with shotcrete).

Figure 1. Standard ground support patterns (as supplied by St Barbara Mines Ltd).

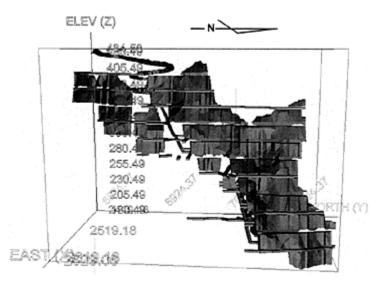


Figure 2. Longsection of Gibraltar mine.

Prediction of the shear location assisted the planning of the development schedule for the contractor. Three of the four intersections with the decline required additional ground support.

Development of the 400 mRL crosscut intersected a sub-parallel $(70^{\circ}/078^{\circ})$ shear with a plan horizontal width of approximately 30 m (true thickness ~24 m). This crosscut was abandoned and the decline continued to the 380 mRL. In the ventilation drive a new shear was intersected. This time the orientation was 53° towards 291°. Examination of the intersection of these two shears gave a northerly plunge of approximately 20°, which coincidentally corresponded to the apparent plunge of the mineralisation to the north.

The decline development continued to 360 mRL, while ore development was commenced on the 380 mRL.

Ore development, where possible, was confined to the quartz carbonate/calc-silicate zone. However difficulties were experienced with the level development due to problems identifying the mineralized zone. The line of the development took on a zigzag direction. Later geotechnical analysis provided a "20–20" hindsight interpretation for the pattern as discussed below.

To this stage, only two dominant shear patterns had been identified. Simple projections of these structures showed that the easterly dipping features could be projected with confidence while the westerly dipping features could not. This was later interpreted to

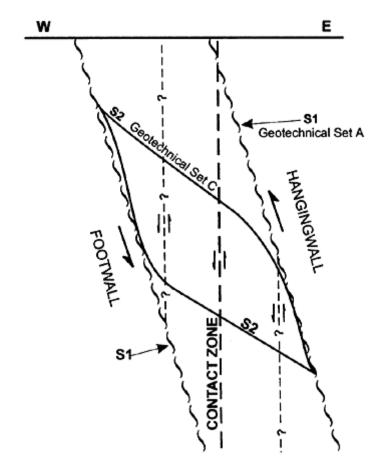


Figure 3. Initial from of tectogenetic model for Gibraltar.

be the result of tectonic interaction between Gibraltar ore body and Set F.

2.2 Experience at Gibraltar

Experience at Gibraltar showed that the point at which to adopt the more intense support shown in Figure 1 was difficult/impossible to recognise at the time, and often it was only after four or more cuts that the need for additional support was realised. By this stage it was often too late to implement a support pattern to prevent collapse which necessitated expensive and extensive rehabilitation. This raised the question of whether the onset of the zones that required additional support could be predicted.

A technique for interpreting the structural regime applicable to mineralised systems, known as Tectogenesis, has been developed by Dr V Bogacz (Bogacz 2002). The technique has been successfully applied to unravelling the structural controls on

mineralization at Telfer, Mt Gibson, Prominent Hill and Uruguay Minerals project in Uruguay to name a few. In essence the method identifies the specific stress regime applicable at the time of mineralisation, and the major tectonic relationships that control the structural setting.

The principal setting identified at Gibraltar was presented to St Barbara Mines Ltd in March 2001 in the feasibility study (Figure 3) (St Barbara Mines Gibraltar Underground Geotechnical Control Plan).

This was developed from a consideration of the early structural mapping shown in Figure 4. The

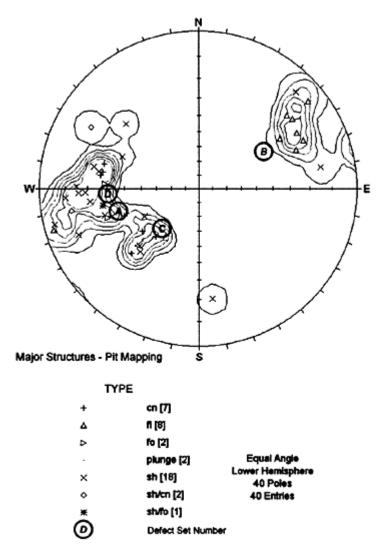


Figure 4. Structural contours of defect data.

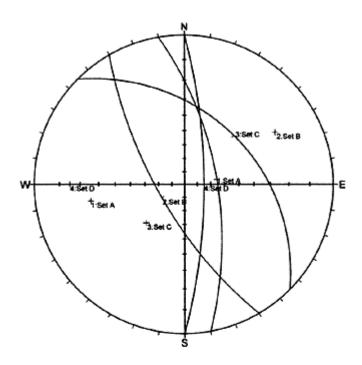
major traces interpreted in that time are shown in Figure 5. Subsequent mapping underground provided additional data as shown in Figure 6. The first collapse of the crown pillar occurred in the north of the pit on 9 April 2002. This was followed by the collapse of the crown pillar in the south (Figure 7) on the 18 April 2002. The prominence of Structure 4 (i.e. Set F) at the time of this mapping was not realised until the collapse of the crown pillar adjacent to a prominent west dipping shear as shown in Figure 7.

These collapses were interpreted at the time and showed that the controlling features were:

Set	Dip/Direction
А	56°/089°
Е	71°/078°
F	53°/291°

Figure 8 shows a 3-D image of the major shears relating to the pit.

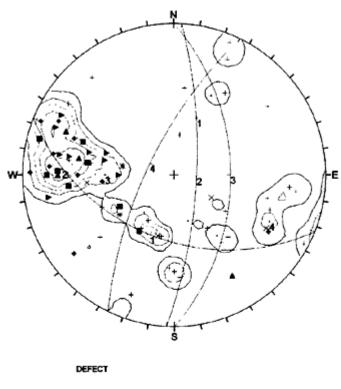
At the same time, a single collapse had occurred underground which did not exhibit any indication of structural problems beforehand. With the experience underground in not being able to anticipate the location of intensely sheared zones which lead to the subsequent instability (most development in the ore was in quartz porphyry), and with the experience with the crown pillar, it was decided to re-examine the Tectogenetic relationships relating to the mineralization to see whether there was any discernable way to predict problem areas.



Average Major Structures & Defect Set Orientations

Or	rientations	
ID	Dip/Direction	
A B	065 / 80 070 / 240	Equal Angle Lower Hemisphere 40 Poles 40 Entries
С	040 / 45	
D	075 / 90	

Figure 5. Principal defect orientations.

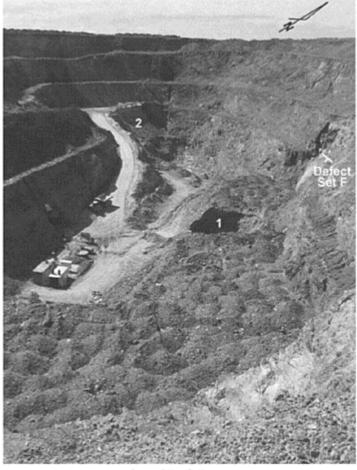


•	CN [8]
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- ▲ CN/SHR [4]
- ► FO/SHR [9]
- JO [35]
- × JOVN [4]
- SHR [10]
- VN[2]

Equal Angle Lower Hemisphere 72 Poles 72 Entries

Figure 6. Data from underground mapping.



Crown Pillar Collapse 1 - In Foreground 2 - In North end of pit

Figure7. Crawn pillar collapse.

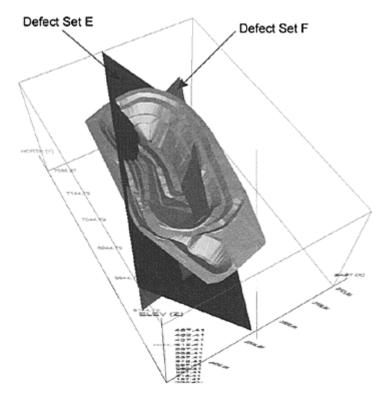


Figure 8. Gibraltar pit showing dominant structures.

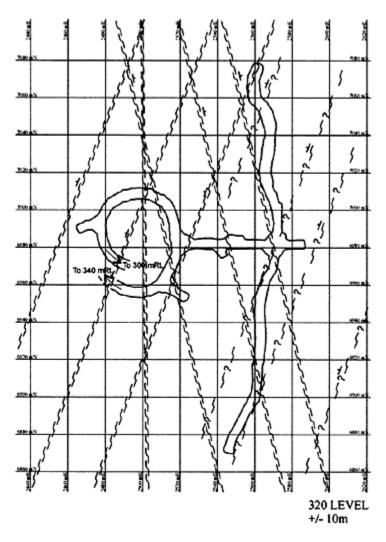


Figure 9. Principal shears influencing underground behaviour.

The first step was to locate the known major structures in 3-D space and to project them through all levels. This involved simplified assumptions:

- 1. The structures were shears and therefore likely to be approximately linear/planar.
- 2. The structures could be projected over lengths of 200 m.

3. The dominant structures were:

Set Dip/Direction

A 56°/089°

Е	71°/078°
F	53°/291°

4. Using the layout of the development underground in the ore-drives, an apparent spacing could be derived.

Assumptions 1 and 2 had already been tested by exploratory drilling and subsequent development of the decline, where a shear which intersected the decline was able to be predicted with confidence over several levels.

In order *not* to bias the results (i.e. this was to be a Type 2 prediction after Lambe (Lambe 1973), all the projections were developed before going underground to test the approach.

The structural pattern developed from the Tectogenetic model is shown in plan in Figure 9 at the 320 mRL.

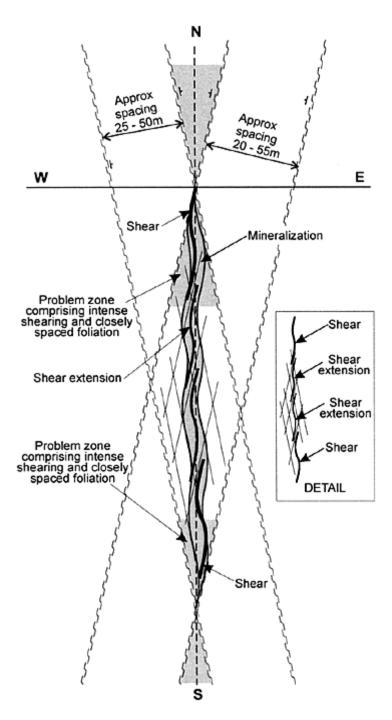


Figure 10. Tectogenetic model in plan.

The traces relate to the different major structural corridors. Examples of the shear, shear extension, and extensional zones are shown in Figure 10.

As noted earlier, the initial projection of Set F did not provide a good intersection with the ore horizons. A review of the projection indicated that it would "step path" through the foliated ground with an apparent dip of $84^{\circ}/282^{\circ}$ which is steeper than Set F. With this resolved, the pattern of simplified projected structures has been reproduced in Figures 11 to 16.

Once the model had been prepared, flitch plans were produced and an underground inspection was conducted to examine whether the Type 2 prediction could be used to identify where problem ground had been encountered. The focus of the investigation was to examine where there was a confluence of the shears as this would coincide with more intense local shearing.

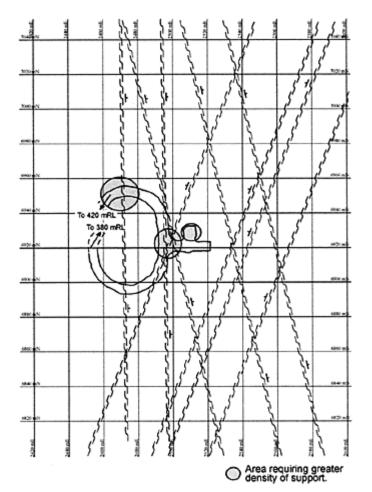
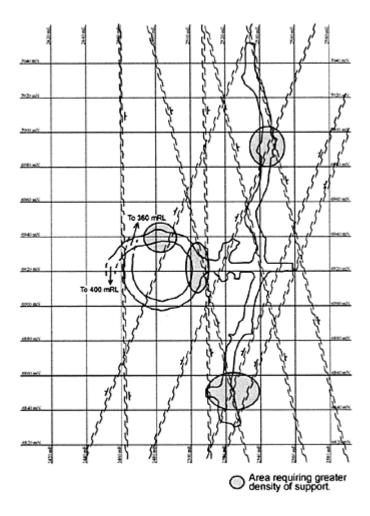
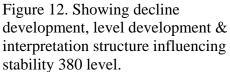
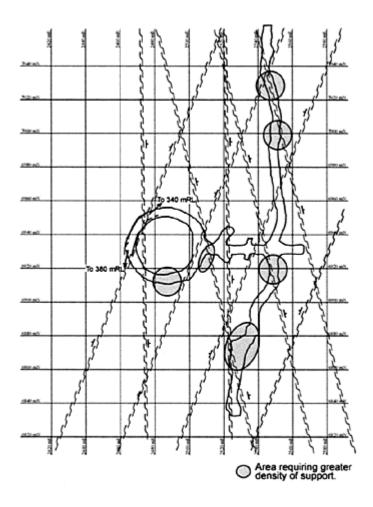
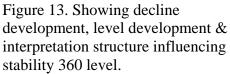


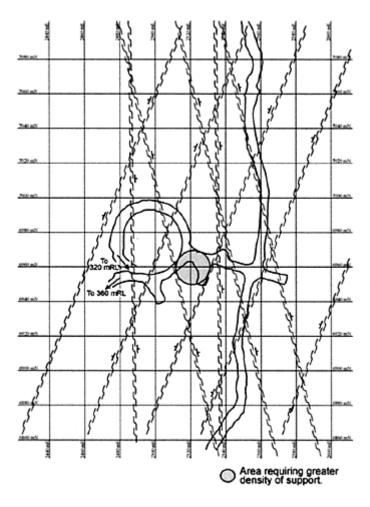
Figure 11. Showing decline development, level development & interpretation structure influencing stability 400 level.

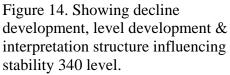












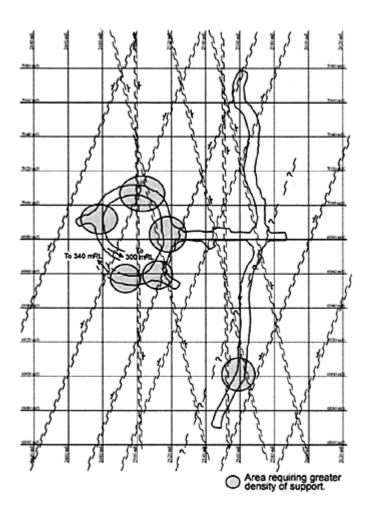


Figure 15. Showing decline development, level development & interpretation structure influencing stability 320 level.

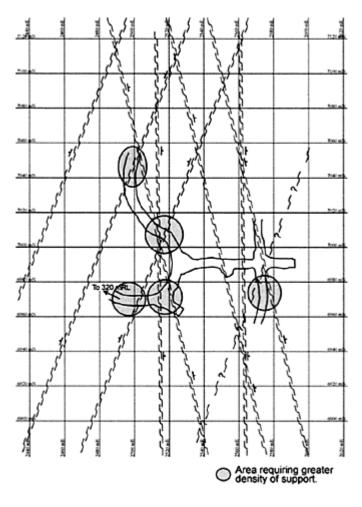


Figure 16. Showing decline development, level development & interpretation structure influencing stability 300 level.

Acknowledging that the model was a gross simplification, all zones that actually required more intense support were very close to the zone of the point predicted. This was at April 2002.

This correlation indicated that the approach could be a powerful predictor of problem areas.

The approach was used by SBML and it assisted in the prediction of zones of potential change. However as a guide it did not prevent further ground collapses. Even with the guide you could still mine 1–3 cuts into structures before it was possible to identify them,

even though they were being looked for at all times. Further failures occurred in ore drives and slope pillars.

A full time geotechnical consultant started work at Gibraltar in June 2002 and remained with the project until completion in February 2003 to provide the detailed interpretation needed to identify problem areas.

3 CONCLUSIONS

Pattern support systems to meet various conditions at mines is an appropriate technique.

The guidelines for application of the support patterns are often extremely difficult to communicate. This is made much worse with staff turnover and lack of continuity/experience with the particular project.

The application of Tectogenetic techniques for this problem showed that the methodology is worthy of consideration.

ACKNOWELDGEMENTS

The support of St Barbara Mines Ltd in allowing the presentation of this paper is gratefully acknowledged. Without the independent verification of Mr Rob Williams, it would be hard to believe that the approach would be as robust as it apparently showed.

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10 Corrosion

Premature bolt failures in Australian coal mines due to stress corrosion cracking

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ABSTRACT: An industry-funded research project in Australia recently investigated the problem of premature rock bolt failures in coal mines. Prior to this study, there had been anecdotal evidence of similar failures from various sites. Sampling and analysis conducted by this project whilst being below a statistically representative sample of the entire industry bolt population—highlighted the fact that bolt failures were continuing to occur at a number of sites. The failures were almost entirely attributable to stress corrosion cracking (SCC). There is no simple solution to the SCC problem, however, as with similar findings from the UK, it appears that steels with higher fracture toughness (Charpy Index) are less susceptible to SCC.

Initial testing was limited to failures below the resin encapsulation. A real concern is the potential extent of the problem at higher horizons. Trials in Australia of a non-destructive ultrasonic device developed by DMT in Germany have proved encouraging. This paper describes the outcomes of the project, completed in 2002, and subsequent experience.

1 INTRODUCTION

1.1 Project background

Underground coal mines in Australia rely extensively on effective ground control through the use of resingrouted rock bolts as the primary form of roof support. Any failure of these bolts can lead to a loss of integrity of roof stability with serious implications for both mineworker safety and the mine operations.

The premature bolt failure project (Crosky et al, 2002), funded by ACARP (Australian Coal Association Research Program), was commenced in 1999 in response to "piecemeal" evidence of rock bolt failures in a number of mines, many in relatively benign loading environments. This led to the recognition of the problem of "stress corrosion cracking", SCC, as a factor in many of these instances. The problem appeared to be linked to characteristics of either the bolt metallurgy or the bolt manufacturing process (or both), coupled with a source of corrosion. The problem had been recognised by various parties in Australia, including the three organisations involved in formulating this research project. Similar experiences had also been reported from the UK coal industry.

Stress corrosion cracking is defined, in layman's terms, as "Stress corrosion cracking is slow, progressive crack growth under the application of a sustained load (either residual or applied) in a mildly corrosive environment, with failure occurring below the ultimate tensile strength of the material".

The Australian coal industry has become very dependent on the reliable performance of rock bolts as a permanent means of ground support. Use of rock bolts for roof control in underground mining in Australia is now the universally accepted primary support practice. Australia has also been one of the international leaders in ongoing development and application of rock bolt-based reinforcement systems within the international coal industry over the last two decades.

Significant trends over the past ten years have included:

- Use of longer bolts.
- Use of partial, and predominantly full-encapsulation, polyester resin anchored bolts.
- Use of threaded bolt fixing systems.
- Adoption of bolt pre-tensioning in an increasing number of applications.
- Adoption of different grades of steel to achieve stiffer and stronger bolts.
- Variations to bolt deform patterns and ribbing systems for improved anchorage and load transfer performance.

In spite of these very positive trends, together with an accompanying significant increase in the level of geotechnical understanding of reinforcement mechanics, there has been reasonably widespread, and increasing evidence, referred to above, of roof bolt failures occurring within the industry. These failures were occurring not only in older bolts, but in some quite recent installations (weeks and months, rather than years after installation). They were occurring both within the resin-grouted horizon, immediately below it, and down to the plate/nut horizon (within the threaded length, on some occasions). In many instances, these failures appeared to be occurring in loading environments considered to be well below the design capacity of the bolting system. SCC was considered to be a likely cause of this problem.

The consequence of these trends, and observed failures, was that the level of bolt reinforcement reliability and capacity was in question in a number of mines both in currently used headings, and older areas of the mines. The level of understanding of the problem at the time was not well developed (in relation to rock bolt performance), with apparent conflicts and debate on the possible contributing issues of metallurgical, manufacturing, loading system and environmental factors. A need was recognised to develop a clearer understanding of both the mechanism and components of the problem, and the extent of the problem, throughout the industry, before steps could then be taken to attempt to minimise, eliminate or control the problem.

Figure 1 shows a typical example of bolts broken by SCC, whilst Figure 2 shows the bolt fracture surface with the characteristic SCC radiating failure pattern emanating from a semi-circular initial crack location.

1.2 Research objectives and methodology

The objectives of the ACARP research project were:

1. To develop a database of rock bolt failure from within the Australian coal industry, with particular

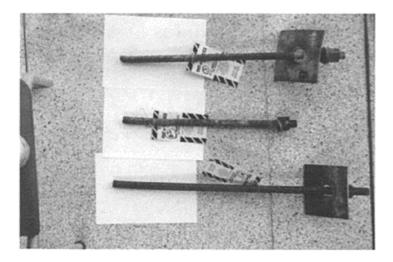


Figure 1. Selection of broken bolts.

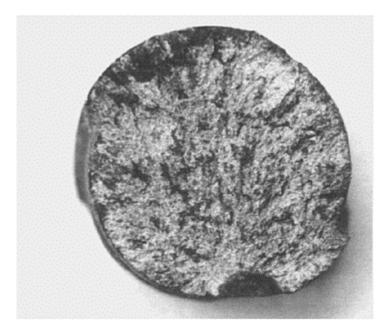


Figure 2. Typical SCC bolt failure surface.

reference to the occurrence, frequency and nature of SCC as a cause of failure.

- 2. To critically analyse the above database with respect to causes of bolt failure with a view to formulating recommendations for future testing programs, design, manufacture and installation procedures to minimize the problems identified.
- 3. To evaluate the available technology for in-situ testing of bolt integrity as a means of identifying up-hole bolt crack initiation and/or failure.

The methodology adopted for the project was as follows:

- Collation of all current observations, data and knowledge of SCC problems, and evidence of broken bolts.
- Establish an initial database of case histories, and conduct metallurgical evaluation of all broken bolt samples collected.
- Develop a more comprehensive database format, for use at an individual minesite where a complete survey of sections of the mine could be conducted to collate information about the mine environment (bolt types, history, loading, geology, groundwater etc.) as well as gathering further samples for met
- Combine all metallurgical data for analysis and allurgical testing. reporting.

The above stages were conducted in series, as sequential steps in the research investigation. In parallel, a number of additional stages were pursued, independently. These were:

• Review UK experience and compare with Australian.

- Source information on non-destructive bolt integrity testing devices, and evaluate for potential use in Australia.
- Investigate existence and nature of bacterial corrosion as a potential contributor to premature bolt failure.

2 FIELD SURVEYS

2.1 Initial survey

An initial survey was conducted across a number of collieries where premature bolt failures had been detected. These collieries covered the major Australian coal basins, and many of the major seams mined by underground methods. The data obtained from this initial survey can be summarised as follows:

- 12 mines were listed in the summary incidents of broken bolts, only two of those mines reported more than a limited number of local failures.
- The failures at all of the mines were limited to specific areas and time over a six year period (1994 through to 2000). With the exception of one mine, no broken bolts installed prior to 1994 have been reported.
- The only failures identified are those that fall out of the roof or that are significantly displaced out of the roof. There is a strong possibility that:
 - Bolts have failed within the encapsulated section of the bolt and remain in place. (Confirmed by at least two incidents of broken bolts exposed in falls).
 - Additional bolts prematurely broken within their free length but in roofs exhibiting shear, do not significantly displace as they are held in place by mechanical interlock.
- The mines showing higher frequency of bolt failure do not achieve full encapsulation and typically are noted in environments that would not have sufficient roof shear to prevent the bolts falling from the roof.
- Although detailed records are not routinely kept, there is evidence that premature failure may occur between weeks and years of installation. The oldest recorded bolts were placed in 1986 with failure probably occurring in the later 1990's. There are other reports of bolts prematurely failing within weeks of installation. (There was no evidence of bolt failures during the installation process).
- As discussed in other sections of the report, the level of visible corrosion of the roof bolts can be negligible at the time of failure of the bolt. This would be consistent with some failures occurring on defects that are either pre-existing in the bolt or else develop rapidly at a localised position on the bolt.
- With the exception of 2 mines the number of broken bolts identified is limited. The two mines where more frequent occurrences were noted both have thick coal roof strata and clay bands are present.
- 4 other mines with similar geology (thick coal or carbonaceous mudstone roof with clay bands) also reported some broken bolts.

- The issue of clay bands and thick coal may be a potential indicator of elevated probability of premature failure occurring however, there are mines in close proximity to those where significant failure has occurred, mining in very similar geology and stress conditions that have reported no failures.
- The clayband/coal lithology is noted for difficulty in achieving full encapsulation due to loss of resin into the strata. Typically bolts in this environment are not encapsulated for up to 500 mm into the roof.
- Groundwater and chemistry issues have been identified as factors influencing the level of corrosion and stress corrosion. This includes alkaline and H₂S environments as well as those that are acidic. Groundwater samples have been collected by the mine sites, but no systematic survey has been carried out.
- The geochemical environment related to clay bands is also conducive to various bacteria that can cause various forms of corrosion.
- Within the limitations of the survey already stated, it is evident that the bolt loading in areas of broken bolts include the possibility of shear with associated bolt bending. This is not to infer that the prematurely broken bolts uniquely result from shear deformation or bending. A percentage of collected bolts do not show any permanent strain that would indicate high bending loads. (Strains that exceed the plastic limit of the steel).
- The presence of clay bands, noted in several failure sites, are commonly associated with shear deformation of the strata. The shear deformation results in bending of the bolt that maybe within the elastic range of the steel. Bending of the bolts would result in higher tensional strains on one side of the bolt balanced by lower strains or even compression on the opposite side. Bolt tensionning may also induce torsional effects in the bolt. Even where the axial load in the bolt is low, insufficient to deform a

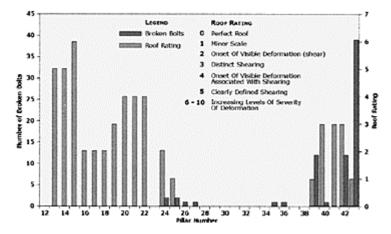


Figure 3. Panel 1: location and frequency of broken bolts.

bearing plate, it is possible to have axial strains close to or even exceeding the plastic limit of the steel resulting from shear deformation.

2.2 Detailed site survey

As part of the project research methodology, a comprehensive database structure was developed to capture a range of potentially relevant data from a site experiencing significant premature bolt failures. A mine site in the Western Coalfield of NSW, at which broken bolts were identified in a number of locations, offered one of the few opportunities to carry out such a systematic survey. The panel that was available for the survey had the following benefits:

- The panel was driven in similar geology throughout its length. The roof strata consisting of interbedded mudstone, carbonaceous shale and coal, with clearly defined and consistent clay bands within the bolting horizon.
- The depth of cover was relatively uniform along its length.

Significant variation of stress field along its length causing variable roof behaviour in zones along its length that are identified on driveage by systematic mapping:

- The panel was driven over a 10 year period, all of it supported with high strength X grade roof bolts or similar, from a range of manufacturers with various steel types.
- The return heading of the panel that was inspected was not used for routine traffic. This meant that failed bolts remained in place or on the floor of the heading and could be tested. (Failed bolts in other trafficable roadways would commonly be removed).

The results of the survey are indicated in Figures 3 and 4. Figure 3 summarises the roof condition noted on driveage of the panel and the number of broken bolts occurring in the 44 pillars of roadway. The highest level of roof deformation and strata shearing

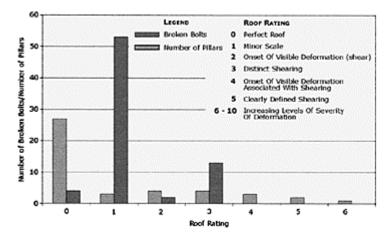


Figure 4. Boltfailuresrelativetoroof condition.

occurred between 14 and 27 Cut-throughs and a zone of moderate deformation between 41 and 43 Cut-throughs. The occurrence of identified broken bolts is limited to sections of roadway between:

- 25 and 29 Cut-throughs
- 36 and 44 Cut-throughs

The broken bolts collected were either composed of non-tempcored X grade steel or AXR type bolts that were also non-tempcored. Typically the bolt segments recovered were between 0.3 m and 0.7 m in length with failure occurring with slight or no bending evident.

No broken bolts were recorded in the initial 23 pillars of driveage. This is consistent with no broken bolts being recorded in the adjacent extraction panels (Longwall B and Longwall C) that were driven in this time period. The broken bolts identified in the 25 to 29 Cut-through section would be consistent with broken bolts recorded in the adjacent Longwall D and E extraction driven during the same period.

The high frequency of broken bolts recorded in the 36 to 44 Cut-through section of the panel would be consistent with broken bolts recorded in other driveages including Longwalls F–G gateroads and the driveage of Panel 2 which is adjacent to 10 to 12 Cut-through in the main survey heading.

Figure 4 indicates the relationship between the number of broken bolts noted and the roof condition rating on driveage. It shows that no broken bolts were noted in highly deformed roofs (category 4 to 6) and the majority of the broken bolts 53 out of 72 (74%) were located in Category 1 roof. This category is for roof strata with low levels of visible deterioration but with minor scaling.

3 METALLURGICAL INVESTIGATIONS

3.1 Testing program

A total of 44 failed rockbolts recovered from four separate collieries were examined. The work involved examination of the fractures, magnetic particle inspection for cracking, chemical analysis, Charpy impact testing, hardness testing and microstructural evaluation. Longitudinal strain measurements were also made on 11 of the bolts. The type of bolt was identified, where possible, from the rib pattern. The bolts included HPC (3 bolts), Threadbar (3 bolts), Wriggle (1 bolt), X (15 bolts), AVH (12 bolts), AXR (8 bolts) and X grade Tempcore (2 bolts).

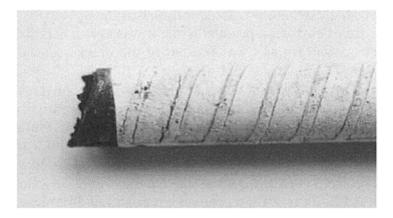


Figure 5. Additional cracking adjacent to the fracture surface.

It was found that the bolts had fractured at various positions along their length with the fracture plane being essentially perpendicular to the bolt axis. All bolts except for one had failed outside the threaded end. A small number of the bolts were distinctly bent. Most of the others showed a discernible bend when examined against a straight edge. In most of these bolts, the bend was near the location of the fracture. Away from the fracture most of these bolts were essentially straight.

Each of the bolts was inspected for additional cracking (beyond the actual failure surface) using magnetic particle inspection. Cracks were detected in Bolts 19, 28, 33, 35, 37, 38 and 40. In each case, the cracks were found within 100 mm of the fracture. The majority of the cracks were located near the inside radius of the ribs and all cracks were approximately perpendicular to the axis of the bolt. The cracks were generally on the tension side of the bolts when the bolts were discernibly bent. Bolts 1–18, 24 and 44 were subsequently examined within the region 50 mm from the fracture surface using a low power stereo microscope. Cracks were detected in Bolts 7, 8, 9, 17, 24 and 44. Most of these cracks were again situated around the inside radius of the ribs. In several bolts evidence was found of cracking that had been initiated from the original bolt rolling process during manufacture. Figure 5 shows the additional cracking in bolt 28. Figure 6 shows typical crack branching, characteristic of SCC failure.

Figure 7 shows decarburisation within the cracked region indicating that the crack originated from the bolt manufacturing process.

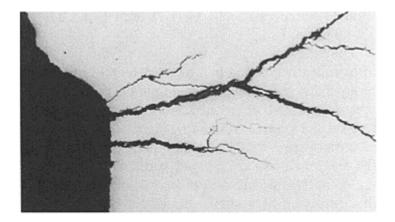


Figure 6. Magnified image of crack branching.

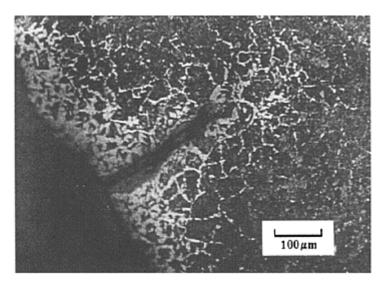


Figure 7. Crack seen in Bolt 44. Note decarburisation alongside the crack.

Chemical analysis was performed on each of the 44 bolts using atomic emission spectrometry (spectrographic analysis). The samples for analysis were taken from the threaded region for Bolts 1–6 and 44, while for the remainder they were taken from the shank of the bolt approximately 70 mm from the start of the thread, unless there was insufficient shank. In these cases, the sample was taken closer to the threaded region. The bolts were found to be of 6 different chemical types. These were:

A. 0.4%C plain carbon steel (Bolt 24)

- B. 0.5%C plain carbon steel (Bolts 4–6)
- C. 0.4%C microalloyed steel (Bolts 1–3)
- D. 0.5%C chromium steel (Bolts 7–10, 12–18 and 44)
- E. 0.55%C manganese steel (Bolts 11, 19–23, 25–27, 29–31, 33–43)
- F. 0.5%C Tempcore (Bolts 28 and 32).

Of the three microalloyed steel (Type C) bolts, Bolt 1 was a high manganese low silicon variant (Type C1) while Bolts 2 and 3 were both a high silicon variant (Type C2). Bolt 1 had unusually high sulphur. Of the 0.55%C manganese steel (Type E) bolts, Bolts 19–23, 25–27 and 29–31 had slightly more chromium and a little less manganese (Type E1) than Bolts 11 and 33–43 (Type E2).

The fracture surface from each of the bolts was ultrasonically cleaned in a water/detergent mixture to remove dirt and loose rust and then examined using a low power stereo microscope. After cleaning, the fracture surfaces varied from being heavily rusted to being essentially rust-free. After cleaning, the majority of the fracture surfaces showed a distinct discoloured region at the fracture origin. In the simplest of cases, this consisted of a small semicircular region which extended inwards from the original surface. The depth of the discoloured region varied from ~1 mm to ~7 mm. In most bolts, at least one of the origins was located at a rib.

Charpy impact tests were performed on all of the bolts except Bolt 44, to determine fracture toughness (a measure of susceptibility to fracture initiation). The test pieces were standard notched 10 mm square test pieces per AS 1544.2–1989, which had been machined from the centre of each bolt. The tests were conducted at 23°C. Thirty four of the forty three bolts tested were X, AVH or AXR and these all had low impact energies. The X and AXR bolts had the lowest impact energies, all being in the range 4–6 Joules. The impact energies for the AVH bolts were marginally higher, being from 5–8 Joules. In contrast, the two Tempcore X bolts had impact energies of 14–16 Joules while the Wriggle bolt had an impact energy of 20 Joules. One of the HPC bolts (Bolt 1) had an impact energy of 14 joules, but the other two (Bolts 2 and 3) had lower impact energies of 8–9 Joules; however, the composition of Bolt 1 was slightly different to that of Bolts 2 and 3.

3.2 Discussion

The 44 broken bolts that were examined were of eight different types and included at least six different chemical compositions, indicating the widespread nature of the failures. However, most of the failed bolts showed a set of similar features, these being:

- an absence of significant necking in the vicinity of the fracture,
- discernible bending in the vicinity of the fracture,
- a fracture surface perpendicular to the axis of the bolts,
- a discoloured region at the fracture origin.

The discoloured region seen at the fracture origin is consistent with failure having been initiated by stress corrosion cracking. Failure by stress corrosion cracking involves the joint action of stress (applied and/or residual) and a corrosive medium. A consequence of this process is that the fracture surface becomes coated with corrosion product. SEM examination of the discoloured region at the fracture origin in Bolt 44 confirmed that the discolouration was due to the presence of corrosion product while it was also noted that corrosion product was present in the cracks seen in Bolt 40 right up to their tips.

Stress corrosion cracking occurs by slow progressive growth of the stress corrosion cracks. Eventually one of the cracks will reach a critical depth at which the remaining section can no longer support the load and rapid overload failure then occurs. The fracture produced during this final stage in the failure does not involve action of the corrosive medium and the fracture surface will not be corroded (unless it remains in contact with the corrosive medium for some time after failure has occurred). The fractures seen on the rockbolts generally showed minimal corrosion outside the discoloured origin region, consistent with this mode of failure.

Further confirmation that the fractures were produced by stress corrosion cracking was obtained from the metallographic sections made through some of the additional cracks seen in Bolt 40. These were found to be extensively branched, as is characteristic of stress corrosion cracking. The presence of multiple cracks, as seen on the surface of some of the bolts, and as the multiple fracture origins in many of the others, is also characteristic of stress corrosion cracking.

In most cases at least one of the crack origins was located at the base of a rib. Likewise, the additional cracks detected near the fracture in some of the bolts were usually located along the base of the ribs. Since the ribs acts as stress concentrators, the development of stress corrosion cracks would be expected to occur preferentially along their base. The cracks in the surface of Bolt 40 were seen to be restricted to one side of the bolt, this corresponding with the tension side of the bend in the bolt. This indicates that the stress responsible for the stress corrosion cracking in this bolt was produced as a result of the bolt being bent and, thus, that the stress corrosion cracking was initiated by the bending. Most of the other bolts also showed bending and the fractures generally initiated from the tension side of the bending. It is noted that the axial prestress on the bolts would also contribute to the stress responsible for stress corrosion cracking since this would add to the stress produced by bending. As in the UK bolts (Shutter et al, 2001), it is probable that the resin grouting undergoes fracture when the bolts are bent providing access of the corrosive medium to the bolts.

Apart from the bending in the vicinity of the fracture, most of the bolts showed minimal plasticity prior to fracture, this being evident from the minimal reduction in area at the fracture and the lack of tensile elongation. The absence of plasticity is to be expected when failure occurs from stress corrosion cracking.

3.3 SCC behaviour and potential controls

For stress corrosion cracking to occur all of the following factors must exist simultaneously:

- a susceptible material,
- a corrosive environment,
- an applied and/or residual stress.

Removal of any one of these factors will eliminate the problem. There are thus a number of different ways in which the problem may be overcome, or at least alleviated, some of which are:

- reducing the strength of the bolts,
- increasing the toughness of the bolts,
- reducing the pre-stress on the bolts,
- galvanising the bolts,
- cathodically protecting the bolts,
- use of a corrosion inhibitor.

Anecdotal information indicates that failure of rockbolts in Australian collieries became increasingly prevalent as the strength level of the bolts was increased (Doyle, 1999). This is consistent with the observation that susceptibility to stress corrosion cracking increases with the strength level of the material (Ciaraldi, 1992). Thus a reduction in the strength level would be expected to reduce the problem.

Increasing the toughness of the bolts would also aid in reducing the severity of the problem. This would increase the depth to which the stress corrosion cracks can grow before initiating overload failure and thus increase the life of the bolts. It is noted that a very substantial increase in toughness was obtained in the UK bolts by careful control of the composition and processing of the rockbolt steel (Barratt, 1998). Some improvement may be possible for the Australian bolts using a similar approach. However, the level of improvement is unlikely to be as marked as in the UK since the Australian bolts have a higher carbon content (0.5% compared with 0.3% (max.) for the UK bolts) which makes them inherently less tough. Increased toughness could however be obtained by altering the microstructure of the bolts.

Some benefit may also be obtained by reducing the pre-stress on the bolts. This follows since the time to failure by stress corrosion cracking increases markedly as the stress level is reduced (Jones & Cricker, 1992). It was suggested in the UK that galvanising be employed for bolts exposed to more aggressive environments (Shutter et al, 2001). This may also be appropriate for the Australian rockbolts since it would not only protect them from corrosion, but also from stress corrosion cracking. There was concern in the UK that the galvanising process may cause hydrogen embrittlement, but it was demonstrated that this did not occur. However, the Australian rockbolts may be more susceptible to hydrogen embrittlement than their UK counterparts because of their higher carbon content. It is therefore recommended that embrittlement testing be undertaken if galvanising of the bolts is considered as an option.

Similarly, application of a cathodic potential to the rockbolts would stop both corrosion and stress corrosion cracking. Corrosion and stress corrosion cracking can also be stopped by using a suitable corrosion inhibitor. In the context of rockbolting, it may be possible to add a leachable corrosion inhibitor to the resin.

The various solutions given above, while all viable from a fundamental standpoint, would need to be examined carefully before being implemented. In most cases, substantial laboratory testing followed by field trials would need to be undertaken. Some of the solutions may prove to be prohibitively expensive while others may not be suitable in an underground mine environment. Shutter et al (2001) also confirm that the UK rockbolts are controlled by a national specification whereas no national specification

exists in Australia. A performance based specification, as is being introduced in the UK, would seem to be equally useful in Australia. In particular, a minimum toughness should be specified.

4 COMPARISONS WITH UK EXPERIENCE

Premature failure of rock bolts has been known in the UK mining industry for some considerable time, and over the past 8 years the Health and Safety Executive (HSE) has been working with the mining industry, steel manufacturers and with rock bolt suppliers to address this problem.

In summary, this research work in the UK into premature rock bolt failure has produced the following findings:

- most failures were associated with bends in rock bolts;
- all failed rock bolts had corroded to some degree;
- fracture had initiated at the root of a corrosion pit (as opposed to a crack);
- tests also showed that stress corrosion cracking may occur only if hydrogen sulphide is present;
- the minimum depth of corrosion pit that was needed to cause brittle failure of the original bolts was found to be 1 mm;
- the failures had occurred without noticeable ductility and the failure surfaces were brittle in nature;
- in certain conditions, particularly where sharp profiled corrosion pitting is present, the low fracture toughness can result in brittle fractures at about yield.

It is relevant to briefly compare these UK findings with the findings from the present study of premature failure of rock bolts in Australian coal mines. Some key differences should be noted. It should also be noted that these comments are based on a wider database of bolt failures and experience, beyond that investigated in detail, as reported earlier in the paper. The Australian findings can be summarised as follows:

- most failures of rock bolts have occurred in bolts which were straight over the majority of their length;
- not all failed rock bolts were corroded on the bolt surface;
- the initiation point of failure varied from the base of a corrosion pit, the base of a stress corrosion crack, the base of a pre-existing crack, the base of the rib/core junction, and the root diameter of the bolt threaded section;
- premature bolt failure may occur in a number of different ways and is not restricted to typical stress corrosion cracking modes of failure;
- the depth of a pre-failure crack in the bolt prior to ultimate brittle failure varies widely from less than 1 mm to over 40% of the sectional area of the bolt;
- most failures had occurred without noticeable ductility and all the failure surfaces were brittle in nature;
- most brittle failures appear to have occurred at a load which was less than the yield capacity of the bolt (based on the deformation of bearing plates).

4.1 Higher fracture toughness steels

In the past, steel grades used for rock bolt manufacture have been defined by three parameters only, namely ultimate tensile strength, yield strength and elongation. These three parameters can be determined by a simple tensile test, and most steel production for rock bolts provides this data. The previous steel requirements for rock bolts were simply to try and provide both high strength and high elongation, such that the rock bolt could provide high strength and still withstand significant deformation due to roof movement.

Typically, X grade type bolts provided tensile strengths in the range of 30–35 tonnes with elongation values in the range of 8–15%. Lower strength bolts had tensile strengths in the range of 20–28 tonnes, but with a corresponding increase in elongation in the range of 18–25%. There is therefore a trade-off between high strength and high elongation. Nevertheless elongation only indicates the ductility of the steel, and does not provide a quantitative indication of fracture toughness.

Following detailed metallurgical testing of the most common rock bolts used in the Australian coal mining industry as part of this project, it became clear that most high strength rock bolts have been made from steels with a low fracture toughness. This study has revealed that premature failure of rock bolts commonly occurs in steels with a low fracture toughness.

The fracture toughness of steels can be determined by a test as outlined in British Standard BS7448. However, since this fracture toughness test is expensive and difficult to undertake, actual fracture toughness is rarely measured, and usually only a test which indicates fracture toughness, called a Charpy test, is performed. The Charpy test involves machining a sample of steel from a steel bar with a groove machined into it, and then subjecting the sample to an impact load from a pendulum. The amount of rebound of the pendulum indicates the amount of energy absorbed by the sample, and this is the Charpy impact value expressed in joules. For rock bolts, approximate impact values of less than 10 are low, between 10 and 20 are just moderate, between 20 and 30 are good, and above 30 are very good.

Tests undertaken as part of this study indicate that most rock bolts used in the Australian coal mining industry had Charpy impact values of less than 10, and no test results were above 20. These results covered a range of rock bolts and steel from two different steel suppliers.

Consequently, steel manufacturers in the UK, and subsequently in Australia, have been looking to develop higher fracture toughness steels. The production of a high fracture toughness steel requires the development of a fine, even grain structure throughout the steel, and this can be achieved by the inclusion of vanadium or other micro-alloying elements and/or by controlled cooling of the steel.

In the UK, the original Co-Steel steel (Type 1) had Charpy values in the range 4–5 joules. The modified Co-Steel steel (Type 2) has Charpy values in the range 28–0.

In Australia, the original high strength rock bolt steel grades (1055, 1352, 1355) were 0.5 Carbon steels and achieved high strength but had low Charpy impact strength (typically 4–7) which is very similar to the original Co-Steel Type 1 steel grade.

Recent developments in Australia, have seen the production of higher fracture toughness steels both from OneSteel (840 grade) and from Smorgon Steel (15M25, 15M30, 15M35 grades). These higher fracture toughness steels have lower Carbon (typically 0.2 to 0.4) but have some micro-alloying, and/or some controlled cooling of the

steel. The Charpy impact strength from these higher fracture toughness steels varies from 12 to 40 depending on the production batch and the steel producer.

However, these higher fracture toughness steels can have impact values which vary by as much as 10 Joules from the same steel batch. This variability may be due to a number of factors including where the sample was machined from in the bar, the temperature of the test, and the time the test was performed after rolling the steel.

The Charpy test therefore only provides a guide to the actual fracture toughness of the steel, not the actual fracture toughness. Therefore a specification that calls for all rock bolt steels to have a minimum impact value of 23 (as per the UK guidelines), may mean that the average impact value will have to be over 30.

Nevertheless, it is also important to emphasise that although higher fracture toughness steels can be manufactured for rock bolts, it requires the use of expensive alloying elements and/or the use of accelerated cooling or other production refinements which all add to steel production costs. The high fracture toughness steels developed in the UK contained over 0.2% vanadium, and this would add significantly to steel costs.

In summary, the development of high fracture toughness in steels is dependent upon having a fine, even grain structure throughout the steel, and this can be achieved by a combination of alloying elements and the rate of heating and cooling of the steel. However, the consistency of achieving high fracture toughness can be highly variable, depending upon the metallurgical approach taken, and this needs to be considered by the steel makers.

In addition, there is some concern over the reliability of the Charpy test particularly when performed on steels which have been subjected to accelerated cooling. Reports from OneSteel indicate that variations in the Charpy number can be as much as 10 Joules from samples machined from the same bar or the same rolling batch. Clearly there may be some concerns here with the test method and the variability of the results, and this needs to be resolved in any future research.

As indicated above, the Health & Safety Laboratory in the UK has recommended a minimum impact value of 23 be adopted for rock bolts, based on a critical defect size of 3 mm (McGuinness, 2000). However, bolt failures have occurred in Australia where the critical defect size has been as high as 40% of the surface area of the bolt, and as small as 1 mm deep. Clearly, there are some complicating issues concerning premature failure of rock bolts in Australia.

Specifically, the effects of shear loading on bolts needs to be considered. Although most failed bolts that were recovered were straight, this does not preclude them being subjected to shear loading. SCT Operations Pty Ltd (2001) has conducted many tests on rock bolts subjected to shear loading. These tests have demonstrated that shear loading causes one side of the bolt to be in compression and the other side of the bolt to be in tension, and this may occur without a high axial load being generated in the bolt. This may explain why the collar loads on bearing plates from failed bolts are normally very low.

In addition, these localized compressive and tensile stresses may be very high, but the average yield strength of the bolt still may not be exceeded. In this case the permanent deformation in the bolt will be low, and failed bolts may be substantially straight. Moreover, as pointed out in the field survey results, it is straight bolts which are most likely to be recovered, either by simply falling out of the hole or by being able to be

pulled out of the hole. Bolts subjected to substantial shear and bending, are likely to be locked into the borehole even if the bolt has failed, and are therefore less likely to be identified or sampled in any field survey. Shear loading rather than tensile loading of the bolt, appears to be the only rational explanation for the very large variability in crack size prior to ultimate failure of the bolts.

Fracture toughness itself should also be considered. As mentioned previously, indications of fracture toughness using the Charpy test require that a machined sample be obtained from the centre of a bar. For bars with uniform properties this is a valid procedure, but for bars which have been quench and tempered or accelerated cooled, this is not a valid procedure.

Previous Charpy tests on Tempcored bars indicated Charpy values of about 12 to 15. Subsequent Charpy tests on OneSteel's accelerated cooled 840 grade of steel, indicate variable results ranging from about 14 to 30. To date, there have been no confirmed premature bolt failures with the 840 grade of steel.

In summary, there is evidence from the UK, and anecdotal evidence from Australia using higher fracture toughness steels for a short period of time, that higher fracture toughness steels reduce the incidence of premature bolt failures. What is still uncertain is the level of fracture toughness required in rock bolt steels for Australian conditions.

5 BACTERIAL CORROSION

Premature failure of steel components through the action of bacterial corrosion has been noted in many industrial applications. A survey was commissioned by the Project team to investigate the potential occurrence of sulphur and iron bacteria. The survey was carried by the ANSTO—Lucas Heights Science and Technology Centre (Holden & Russell, 1999).

The survey took the form of a non-systematic preliminary investigation designed purely to ascertain whether bacteria were present in mine sediments. The survey was carried out in Gateroad A at Colliery 8 and encompassed an area of known broken bolts. Figure 8 indicates the correlation between bacterial location and broken bolts.

In summary, the report delineates the presence of a wide variety of bacteria that are associated with corrosion of steel in all but one sample collected. These include both iron and sulphur oxidising bacteria.

The survey indicated that the bacteria can be present in large enough quantities to promote significant localised corrosion and or embrittlement. Of interest to the analysis of groundwater acidity influences, the report states *"The fact that the bulk water phase flow-ing through the mine is neutral pH does not preclude"*

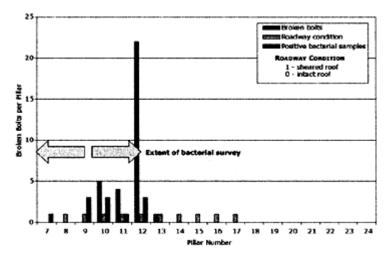


Figure 8. Gateroad A survey—location of positive bacterial samples.

acid corrosion occurring in proximity to the bolts, as it may happen on the micro scale".

Although only one mine site was surveyed the geological environment is not unique and so it could be expected that the bacteria associated with corrosion of steel could be widespread in certain mining areas, specifically those with pyritic clay bands.

When the locations of sites where bacteria were found are compared with the broken bolt survey from gateroad A, it was seen that the bacteria were found throughout the survey area that encompassed the locations of broken bolts. It is possible that higher concentrations of bacteria may influence the frequency of bolt failure; however other factors additional to the presence of bacteria could also be controlling the frequency.

6 NON DESTRUCTIVE TESTING

A literature review revealed that a variety of roof bolt length devices were available including devices manufactured to IS specification. The principal mode of operation of most of the devices was through the use of ultrasonic energy reflected from the failure surface.

Examples of this type are:

Geodynamic AB Sweden
Raymond Engineering (US) USBM device
UK ultrasonic tester
Krakow Mining Academy Poland
DMT Germany

All of the above devices would allow the determination of the actual bolt length of an installed bolt and allow identification of fractured bolts. (Subject to curvature of the bolt itself—that would not enable an adequate signal reflection).

Of the devices investigated the KGH and DMT devices offered the benefit of not needing surface preparation of the end of the roof bolt. The ability to



Figure 9. DMT ultrasonic testing device (*after Ruppel & Wittenberg* (2001)).

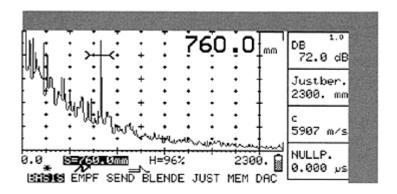


Figure 10. DMT test result indicating bolt failure at 0.76 m (*after Ruppel & Wittenberg (2001)*).

carry out tests on installed bolts without the requirement to grind the end surface of the bolt was considered very beneficial for large scale surveys to be carried out efficiently.

Both the KGH and DMT devices were inspected in operation and would be easy to use within Australian mines subject to IS approval or exemption and are both manufactured to EC IS regulation. The DMT device is now in commercial manufacture while the KGH device was available only through the Academy (at least at the time the project was carried out).

The DMT device (Ruppel & Wittenberg, 2001) indicates one of three states either:

- the bolt length is as expected;
- the bolt length is shorter than expected and estimates the actual length;
- the signal is not sufficiently clear to establish the length. (Excessive curvature or plastic shear of greater than 10 mm).

Figure 9 shows the DMT device, whilst Figure 10 is a result from an underground bolt trial.

With the findings from the mine database and the detailed mine survey it was considered essential that additional surveys be carried out using a device such as that mentioned as it is highly probable that there are a significant number of bolts broken either:

- Within the encapsulated length (two occurrences already noted).
- Broken but held in place by shear action within a deformed roof.

Without this information, current assessment of the extent of premature failure is likely to be highly biased.

7 RECENT PROGRESS

Since completion of the ACARP project in April, 2002, further activity has been limited, due to lack of availability of funding. However, a number of further actions have been taken and outcomes noted, which suggest that ongoing work in this area is warranted.

Firstly, a series of trials of the DMT device in Australia have been conducted. These have included both a colliery where bolts were installed over ten years ago, as well as one where the bolts were installed within the last three years. Neither of these collieries were part of the original survey. In both of these cases, the DMT device provided a clear signal indicating broken bolts present within the roof, within the encapsulation horizon, at heights well short of the installed bolt length (e.g. 0.7 m, for a 2.1 m length bolt). In the case of the newer installation, a complete pillar length of bolts was tested and a significant proportion of the pillar length was found to contain broken bolts. Unfortunately it was not possible to proceed with the ultimate confirmation of these results by overcoring the "failed" bolts.

Secondly, one of the collieries that had experienced premature bolt failures attributed to SCC from the previous survey has subsequently changed to a higher fracture toughness steel in their X bar bolts. Since that time, over an 18 month period, there has been no confirmed reports of any bolt failures—an encouraging sign and one that supports the view that higher fracture toughness steel will assist in reducing, if not preventing SCC failures.

8 CONCLUSIONS

The findings from this project indicate that there are still significant problems in defining the scale of prematurely fractured bolts in coal mines.

The surveys undertaken have indicated (within the limits of the scale of investigations) that:

- Within any given environment the noted occurrence of prematurely failed bolts is typically localised in clusters. Individual clusters can include a high frequency of broken bolts.
- There is general correlation between clusters where the roadway development occurs at the same period.
- On the basis of qualified data collected, the presence of clay bands within the bolting horizon, particularly in coal or carbonaceous mudstones, appear to have the greatest propensity to generate premature failures. This may also, however be a function of difficulty in getting full encapsulation in these environments due to resin loss.
- In the detailed survey, no prematurely broken bolts were identified in roadways driven prior to 1994 and no bolts were identified in high deformation sections of the panel (where the bolts may be failed, but are remain locked into the deformed strata).
- There appear to be general factors that can contribute to the risk of premature bolt failure occurring. These factors are only indicative and may be interrelated. A summary of these factors is:
 - The presence of clay bands within the roof strata.
 - Thick coal roof sections.
 - Limited shearing within the strata inducing bending within the bolts.
 - High tensile steel bolts are used.
 - There is some groundwater present in the strata.
 - The presence of bacteria to promote localised corrosion or growth of already present flaws within bolts.
- Until a more comprehensive investigation in-situ is made of installed bolts to determine whether premature failure has occurred then there is limited value in further analysis of environmental influences.

The metallurgical investigations revealed:

- The latest developments of higher fracture toughness steels in Australia has produced variable fracture toughness results as measured by the Charpy test. This may be due in part to actual variability in the steels but also the inability of the Charpy test to adequately measure fracture toughness on quench and tempered or accelerated cooled steels. Alternative indicators of fracture toughness for rock bolts should be considered (e.g. bending tests, drop tests etc.). Anecdotal evidence and the latest use of higher fracture toughness steels by mines, indicates that higher fracture toughness steels are less susceptible to premature failure than steels with low fracture toughness. Tempcored steels and accelerated cooled steels in particular, appear to reduce the incidence of premature failures. However, the level of fracture toughness required to prevent premature bolt failures requires further investigation.

The project identified the need for non-destructive testing equipment to identify potentially failed bolts within the encapsulation horizon. In terms of available technologies, it was found that:

- Commercially available devices suitable to determine prematurely broken bolts in-situ are available and reasonably well proven. These devices are considered to be the only method to fully assess the extent of premature failed bolts occurring in the industry. Current survey techniques relying on visibly displaced bolts are almost certainly presenting a limited and potentially biased perception of the extent of the problem.
- For this purpose, the device developed by DMT in Germany appears to be one of the most suited and available systems to meet the current needs of the Australian coal industry.
- Further developments of this and other devices will provide greater confidence in the application of such systems, and potentially further increase the capabilities.

In summary, the critical issues that should be addressed in the future by the industry include the following:

- Comprehensive field evaluation of the DMT developed ultrasonic NDT bolt testing device (or similar), to both prove the device for routine industry use, and further quantify the extent of the in-situ bolt integrity problem.
- Further development of metallurgical and corrosion surface test procedures and database expansion—in particular, to develop and test a simplified steel "toughness" test.
- Investigation and documentation of the properties of new steel products in the marketplace.
- Investigation of the extent of potential bacterial "bug" corrosion of bolt steel and possible remedial actions.
- Documentation of the extent of brittle failure of bolts in threaded sections, especially with regard to bolts used for hanging structure, monorails, etc.
- Provision to the industry of guidelines for minimizing SCC problems including bolt and steel traceability.
- Consideration should be given to the provision of an Australian performance standard for rock bolts. In particular, a minimum toughness level should be specified. It is essential that this Standard be based on the key recommendations of this report, including the recommended future research. There should be input from the steel manufacturers, bolt producers and bolt users in the formulation of the standard.

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The views expressed in this paper are those of the authors alone, and do not necessarily reflect the views of the industry as a whole, or of parent or collaborating organisations.

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The corrosion of rock bolts and cable bolts

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ABSTRACT: In this paper laboratory corrosion tests and their results made on rebars and steel strands are presented. The aims of corrosion research were to determine the differences in corrosion rate between solid rebar and cable, to determine the effectiveness of corrosion prevention methods in relation to cable operating time and to find a corrosivity classification method, which could be used in estimating the corrosivity of the environment, where rock bolts are used. The results include weight loss tests, strand structure tests and stress corrosion cracking tests. The effect of the corrosion protection, like epoxy-coating and hot dip galvanizing, on the axial behavior of the steel strands are also discussed on the basis of the laboratory pull out tests made on test samples. The research work was done at the Helsinki University of Technology. The axial load displacement behavior of test bolts was tested by double pipe pull tests with different embedment lenghts. Five different bolt types were tested: rebar, black steel strand, hot dip galvanized steel strand, epoxy coated steel strand and bulbed strand. All the strands were seven-wired steel strand. The rebars were tested as a reference test bolt of the bolt type most used in civil rock engineering.

1 INTRODUCTION

The corrosion of rock bolts has not been researched to date as intensively as the corrosion of concrete steel. The corrosion process of grouted rock bolt is more complicated than that of concrete steel and thus worthy of further research.

Corrosion of the rock reinforcement element can be a problem in long-term reinforcement depending on the corrosivity of the environment. The main purpose of rock reinforcement is to provide safe working conditions during the construction operation and maintain the rock construction safe for its lifespan. This requires good understanding of the corrosion sensitivity of the reinforcement systems in different environments.

2 ROCK BOLT CORROSION

The corrosion process of a rock bolt is very complex and its mechanisms and rate are affected by many factors, which also change during the corrosion process. There are also different forms of corrosion. The main factors are (Sundholm & Forsén 1993):

- Oxygen
- Other gases (carbon dioxide, sulfur dioxide and sulfur trioxide)
- Salts dissolved in water
- pH value
- Organic compounds
- Rate of water inflow and velocity of the flow
- Humidity conditions
- Temperature
- Pressure
- Joints in the rock mass
- Consistency of the water
- Conductivity of the rock types
- Purpose of the use of the excavation (water tunnels and storage, waste water tunnels, fuel storage, gas storage, traffic tunnels).

Careful storage and treatment of the steel strand is essential for achieving successful cable bolt reinforcement. Dry corrosion is always formed on the surface of the cable bolt in long-term storage of the bolts. It is caused by the reaction between iron atoms and atmospheric oxygen. This surface corrosion affects the bond strength of the cable bolt depending on the amount of corrosion, but as long as it is only minor corrosion, no problem arises in cable bolting (Hutchinson & Diederichs 1996). Heavy surface rust is usually the result of exposure to moisture and has a significant influence on the performance of the cable bolts. The corrosion product fills the flutes of the cable preventing the penetration of the grout in installation and seriously reducing the cable/grout interlock, resulting in less cable bond strength. The corrosion also reduces the cross-sectional area of the bolt, reducing the tensile capacity.

The cable bolt material and plates, wedges and other surface fixtures can induce aggressive corrosion if they are not electrochemically compatible. The same problem may arise if cable bolts are installed in rich sulfide ore. Acidic water in the rock joint can cause acid corrosion of the cable bolt. This type of corrosion is very dangerous to the cable bolt due to its accelerated rate (Hutchinson & Diederichs 1996).

The susceptibility to all forms of corrosion increases as steel is strained in tension or in shear across a joint in the rock by rock mass movement, or bent by improper plate installation, the susceptibility to all forms of corrosion increases. Any protective surface rust is cracked by such strain, exposing fresh surfaces. This so-called stress corrosion cracking is important because cables will tend to corrode much more rapidly in aggressive environments precisely when and where their mechanical integrity is most tested and most critical. In the case of grouted cable bolts, load concentrations along the cable length are usually related to full cracking and separation across the grout column. This allows a direct and focused attack on the stressed steel by corrosive agents. Stress corrosion is often the final mechanism in cable bolt failure in corrosive environments (Hutchinson & Diederichs 1996).

High carbon steels, the materials used in the manufacture of steel strand, are less corrosion resistant than the steels used in conventional rock bolts. The geometry and structure of the cable bolt may also increase the potential for detrimental corrosion. If water comes into contact with the cable bolt, it can flow along internal channels formed between outer wires and the king wire and cause corrosion. In the laboratory tests, it has been shown, that those channels are very easily blocked by the materials carried by the water (Eloranta 1986). There might be channels between the outer wires and the grout as well due to the shrinkage of the grout resulting the same behavior described above.

3 CORROSION OF GROUT

Concrete provides alkaline environment for steel to passivate. Hydrated cements normal pH value is 12.6. In perspective of corrosion protection the pH value should be between 9.5 and 13 (Tuutti 1982). Corrosion of reinforcement steel is caused by damage of passivation film. Carbonation, chloride penetration or both together are normally the reasons for the failure of passivation film. The passivation layer formation starts with dissolution and formation of Fe²⁺-ions. With adequate dissolution rate and if the growing corrosion products are insoluble, protective layer of iron oxide can form. Formation of passive film also requires sufficient oxygen concentration (Heinonen 2002).

In undamaged correctly installed rock bolt the decisive factor influencing corrosion to start is carbonation of concrete. The carbonation occurs when carbon dioxide reacts with calcium hydroxide. The carbonation rate depends on diffusion of CO_2 and is dependent on (Tuutti 1982):

- The surrounding concentration of CO₂
- The possible absorption of CO₂ in concrete

- The permeability of the material.

Actually carbonation process happens in stages and in several intermediate reactions. If concrete is dry, there is no contact between the anode and the cathode area and no corrosion. However, occasional dry period of time could be harmful because of the more rapid progress of carbonation. (Tuutti 1982) Iron oxide layers cannot protect steel when the pH value goes below 11.5 or the chloride concentration is high enough. The formation of oxides will not end, but their solubility becomes high. Decreasing pH value also makes corrosion by chlorides easier. The value of chloride concentration needed to activate corrosion is dependent of the concentration of OH⁻-ions (Sistonen et al. 2000).

Corrosion rates of reinforcement depend among others on temperature, relative moisture, carbonation and chloride content. In noncarbonated concrete the corrosion rate of zinc is about 1.5 μ m/a. The corrosion rate for steel is about 1.0 μ m/a. After carbonation of concrete, the corrosion rates are increased because of the decrease in pH value. The corrosion rates are about 3 μ m/a for zinc and 22.5 μ m/a for steel. (Sistonen et al. 2000).

4 CORROSION IN CRACKS

In reality, in rock reinforcement, the protective alkalinity of concrete is lost and solution can reach the bolt material. This happens when cracks or spalls occur. For example corrosion products or rock movement can cause cracks. Cracks in concrete cover have three special effects on the corrosion of reinforcement. The first one is that carbon dioxide and chlorides get easier to the reinforcement. Secondly it makes the penetration of necessary substances (oxygen, water) for corrosion easier. Thirdly cracks generate discontinuity to the environment surrounding the reinforcement, creating potential differences in material. Cracks can also generate differences in stresses. Crack in concrete accelerates the speed of carbonation by facilitating the diffusion of CO_2 . The concrete surrounding the crack is carbonated faster than other parts, and this way it causes alkalinity differences on the surface of bolt. In chloride containing solution the initiation of corrosion happens earlier, because of the faster penetration of chlorides. In cracked environment carbonation is slower than the penetration of chlorides. This results from considerable capacity of the concrete for absorbing CO_2 . Corrosion attacks by chlorides also become deep in a short time. (Tuutti 1982)

In narrow cracks repassivation by restored pH value is possible. Corrosion products can fill the crack and keep the neutralising substances out, while alkaline substances from concrete can diffuse to the steel surface. Repassivation seems to be possible if only there are no chlorides present (Sistonen et al. 2000). In rock bolting the thin concrete cover makes repassivation more difficult.

The beginning of corrosion depends on the crack width. Once corrosion has started, the rate of corrosion is not dependent on the crack width. The rate of corrosion depends on the oxygen diffusion on the cathodic areas and electrical conductivity of the path through concrete between anode and cathode. The water content of concrete is a critical factor for both. If the concrete is completely saturated with water, the resistance is low but the rate at which oxygen can diffuse through the saturated concrete is also low. This reduces the rate of corrosion. Again the rate of corrosion is reduced if the concrete is dry. Then oxygen supply is high, but also the resistance is high. The most hazardous environment is when concrete is exposed to alternate wetting and drying. This leads to the optimum moisture being present at least occasionally (Society of chemical industry 1979). It is also said that if concrete is completely under continuous immersion, corrosion cannot occur, because pores are filled with water slowing the oxygen diffusion. In cases like this cracks less than 0.5 mm wide are safe (Tuutti 1982).

5 CORROSION PROTECTION ALTERNATIVES

Cable bolts with multiple corrosion protection have been used in anchoring where the strands are greased and sheathed and grouted inside the single or double corrugated sheathing which is grouted into the borehole. These multiple sheathings together with cement grout provide multiple corrosion protection. In anchoring, the strands are usually pre-grouted into the corrugated sheathing and then the stiff anchor system is transported to the worksite and grouted into the borehole.

Unfortunately, the corrosion protection system used in anchoring is not applicable in rock reinforcement for many reasons. The smaller borehole used in rock reinforcement affects so that it is not possible to install any extra sheathing into the hole. In rock reinforcement, the full length of the bolt is usually utilized for bonding and, thus, no debonding caused by greasing or sheathing is desirable. The grout around the rock bolt does not provide full resistance against corrosion as in an anchor system because the grout annulus around the bolt is smaller, the bolt is not usually centralized and the grout is cracking. The cracking provides channels for water flow especially in discontinuities where the movements of the rock mass take place.

The experience of corrosion-protected cable bolts in rock reinforcement is very limited. The glass fiber cable bolts have good resistance against corrosion, but there are drawbacks like low shear strength and unsuitability for mechanized installation. Stainless steel strand was not available from any manufacturer at the time of this research. Cable bolts can be protected from corrosion by galvanization, epoxy coating or encapsulating. Epoxy-encapsulated steel strands have been used in bridges and in reinforcement of dams but no reference from the reinforcement of rock masses was found. The difference between epoxy-coated and epoxy-encapsulated steel strand is that the latter also has epoxy filling in the internal voids in the strand and is, thus, better corrosion-protected inside the strand.

Galvanization has been used in protecting rebars against corrosion. Galvanization is of use against non-acidic corrosion. At a pH level <7, the rate of zinc corrosion increases and at pH levels <5, the use of galvanized rock bolts is not recommended (Robinson & Tyler 1999, Heinonen 2002).

6 CORROSION EXPERIMENTS

6.1 Corrosivity classification experiments

In this work, we have reviewed methods to classify corrosivity of ground waters. We have assumed that during use, cracking of the grout will happen. This means that the worst-case scenario is that the bolt is fully exposed to flowing ground water. The experiments were done as immersion tests (Table I). Solutions 1–8 were variations of Allard water, which is used to simulate shallow ground water from granitic terrain. Solution 9 was made to simulate the ground water in Olkiluoto disposal level. Solutions 11 and 12 correspond to Pyhäsalmi mine 1410 m and 600 m level. The DIN value is the calculated corrosivity ranking. The lower is the number the more corrosive is the water. DIN value W0 refers to steel and Wd to galvanized steel. Only the solutions number 1, 6, 9 and 12 were used in SCC tests. The solutions 1 and 9 had low corrosivity, whereas solutions 6 and 12 had high corrosivity.

During the corrosivity classification experiments the test samples were steel rebar (\emptyset 20 mm), standard steel strand (\emptyset 15.2 mm), galvanised rebar (\emptyset 25 mm), galvanised steel strand (\emptyset 17.7 mm) and epoxy-coated strand. Mean thickness of the galvanized layer was 155 µm on rebar and 68 µm on strand. The polarisation resistance and corrosion potential were monitored

	pН	Na^+	Ca ²⁺	Cl	Mg^{2+}	HCO ₃ ⁻	SO42-	DIN	value
No		mmol/l	mmol/l	mmol/l	mmol/l	mmol/l	mmol/l	\mathbf{W}_{0}	$\mathbf{W}_{\mathbf{d}}$
1	8.4	2.287	0.285	1.370	0.1	1.487	0.1	-2	3
2	5.5-6.0	2.287	0.285	1.370	0.1	1.487	0.1	-5	-2
3	8.4	15.18	0.285	14.30	0.1	1.487	0.1	-5	1
4	5.5-6.0	15.18	0.285	14.30	0.1	1.487	0.1	-8	-3
5	3.0-3.5	2.287	0.285	1.370	0.1	1.487	0.1	-6	-4
6	3.0-3.5	15.18	0.285	14.30	0.1	1.487	0.1	-9	-5
7	8.4	262.9	0.285	262.0	20	1.487	20	-11	-1
8	5.5-6.0	262.9	0.285	262.0	20	1.487	20	-14	-6
9	7.2	208.1	100.6	413.8	2.3	0.165	0.044	-14	1
10	8.4	2.287	0.285	1.37	5.2	1.487	5.2	-5	2
11	6.7–7.3	31.2	11.2	4.85	0.94	_	14.1	-11	1
12	2.6	40.5	12.7	0.8	34.2	0.8	93.6	-15	-5

Table 1. The composition and DIN value of test solutions. Solutions 1–8 and 10 are based on the Allard water.

during immersion. Weight loss was measured in the end of experiments. Epoxy coated samples were observed with impedanspectroscopy.

Samples used in stress corrosion cracking tests were steel rebar (\emptyset 6 mm) a king wire (\emptyset 5 mm) from standard steel strand (\emptyset 15.2 mm) and from galvanised strand (\emptyset 17.7 mm). SCC tests were done using constant deflection tests with 2-point loading per ASTM G38. The stress was calculated to 85% of yield strength. Samples of strands were immersed along with SCC tests to find out, if there would be crevice corrosion between the wires.

6.2 Corrosion rates experiments

The purpose of this work was to find out the differences in corrosion resistance between the rebar and different type of strands. Rock bolts are used in an environment where natural water can be present. The major components having an effect on water corrosivity, in the type of waters examined here, are the pH value, concentration of aggressive ions (Cl⁻ and $SO_4^{2^-}$) concentration of hardness ions (Ca²⁺ and Mg²⁺) and the alkalinity of water.

The variation of the pH value in Finnish ground water is from average 6.3–6.9 to 2.6 found for example in Pyhäsalmi mine. The average chloride content is about 20–30 mg/1 but values over 4000 mg/1 are found in the area of the Litorina Sea. The large variation

of composition of natural water and the lack of information of the combined effects makes it hard to estimate the corrosivity of the bedrock ground water. The suitability of German standard DIN 50929 (DIN 50929 1985) of water corrosivity was examined as classification method for general corrosion.

Corrosion resistance is a relative term meaning the ability of a metal to perform its specified functions

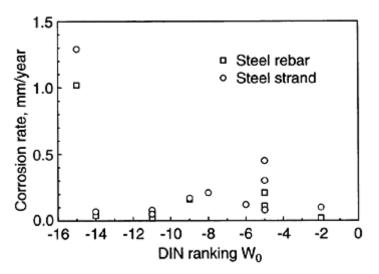


Figure 1. Corrosion rates of steel rebar and steel strand from weight loss measurements.

without impairment due to corrosion in a given corrosion system. In case of rock bolts, the corrosion resistance depends on type of bolt and type of corrosion, e.g. general corrosion, pitting corrosion and stress corrosion cracking. General corrosion proceeds over the whole surface of the metal exposed to the corrosive environment. Localized corrosion is preferentially concentrated on discrete sites of the metal surface exposed to the corrosive environment. Localized corrosion cracking, for example, pits, cracks or grooves.

Stress corrosion cracking (SCC) is a failure resulted by combined effect of specific corrosive environment and tensile stress. In these environments some protective film is present, but when damaged the film rupture initiates localized corrosion. The purpose of the bolting is to support the rock and thus in this case corrosion failure happens, when the bolt cannot stand the load due to decreased area. Use of corrosion protection such as galvanizing or epoxy coating will delay the time when corrosion of bolt material will begin.

6.3 Results

The classification of ground water was done by using standard DIN 50929. The standard classifies water corrosivity using water flow, the position of structure, the concentration of chlorides and sulphides, alkalinity, the calcium content and the pH value. Every factor has specific levels described by a number and corrosivity is calculated using these numbers. Water corrosivities are grouped in to four levels with respect to pitting corrosion and general corrosion of steels and protectiveness of zinc coating. Of these factors increasing alkalinity, calcium content and pH value decrease corrosivity whereas chlorides and sulphides increase corrosivity. Figure 1 shows corrosion rates obtained from corrosivity classification weight loss tests. The samples have not been under mechanical load.

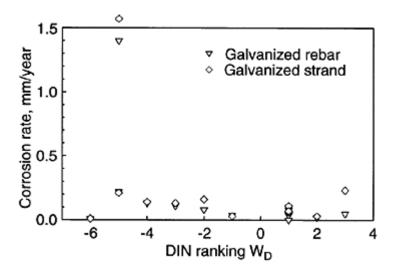


Figure 2. Corrosion rate of galvanized steel rebar and galvanized steel strand versus DIN classification number.

The corrosion rates in Figure 1 and Figure 2 show no regular difference between the rebars and the strands. The general corrosion rate of uncoated rebar and strand is the same as well as those of galvanized rebar and strand. The relation between corrosion rates and DIN values was not clear. When compared to the solution variables, the corrosion rates of steel were found to be dependent on the ratio between aggressive ions and hardness ions. With galvanised steel, the main factor affecting corrosion was the pH value.

The structure tests showed that in test solution No. 1 (Allard water) the steel strand was uniformly corroded. The king wire had corroded the same way as the outer strands. The galvanized strand had much of the zinc layer left. Hardly any corrosion was seen. Calcium carbonate had formed in between the strands. In test solution No. 6 the steel strand had corroded from outer surface. The king wire had not corroded more than outer

wires, but some corrosion product had deposited on king wire. The outer surface of the galvanized strand had corroded. Hardly any zinc was left on the outer wires. The king wire had still zinc left. A large amount of carbonate scale had formed in between the strands.

In test solution No. 9 (Olkiluoto end disposal level) the steel strand was uniformly corroded. A large amount of corrosion products was seen also on the king wire. The galvanized strand had much of the zinc left. Carbonate scale had formed on the outer surface and in between strands. In test solution 12 (Pyhäsalmi mine 600 m level) the steel strand was corroded all over. The king wire had corrosion products on areas that had access to solution, i.e. areas between wires. The outer surface of galvanized strand had corroded. Zinc was left on king wire and on places where the wires had touched each other.

The stress corrosion cracking tests were done with two lengths of immersion. The samples were inspected visually and their corrosion rates were estimated by

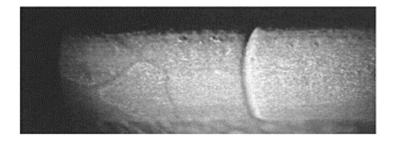


Figure 3. Transverse cracks in galvanized strand in solution 6, with low pH and high chloride concentration.



Figure 4. Longitudinal grooves in steel strand in solution 12, which is very corrosive due to low pH and high amount of dissolvedsalts.

measuring the diameter. Stress corrosion was not detected in the samples. In solution 1, the samples had corroded the same way as unstressed samples. In solution 6, with low pH and high chloride concentration, transverse cracks were seen on both wires, but the samples had not failed (Figure 3).

In solution 6, the stressed rebar had not corroded more than unstressed. In solution 9, all samples showed slight general corrosion. In solution 12, which is very corrosive due to low pH and high amount of dissolved salts, longitudinal grooves were seen in all samples (Figure 4). These are due to texture caused by cold work. The application of tensile stress to 85% of yield strength increased general corrosion rate significantly in very corrosive water 12.

6.4 Results and discussion

The standard DIN 50929 can be used to estimate ground water corrosivity with caution. The correlation between corrosivity ranking and weight loss is acceptable, when the water can be considered as normal ground water. With high level of dissolved solids, the corrosion rate decreases even if the value of the corrosivity ranking is very low. Low pH or dissolved metal ions can increase corrosion rate more than indicated by value of corrosivity ranking.

Excluding the very corrosive solution 12, the corrosion rate of steel rebar varied from 0.02 to 0.21 mm/year and that of steel strand from 0.07 to 0.45 mm/year. The corrosion rates of galvanized rebar and strand were 0.01 to 0.22 mm/year and 0.01 to 0.23 mm/year, respectively. In solution 12, the corrosion rates were in the order 1.0-1.5 mm/year. The corrosion rates should be multiplied by two to get the change in rebar or wire diameter. Mechanical loading used in the SCC tests has doubled the corrosion rate in highly corrosive solution 12, but had no effect in other solutions.

The structure tests indicate that crevice corrosion is not a problem of strands. In test solutions 1, 6 and 9, the carbonate deposit and corrosion products had filled areas between wires. In solution 12, the sample had corroded allover.

Stress corrosion was not detected in the samples. In solution 6, with low pH and high chloride concentration, transverse cracks were seen on both wires, but the samples had not failed. In solution 12, with low pH and high amount of dissolved salts, longitudinal grooves were seen in all samples. These are due to texture caused by cold work.

7 THE AXIAL BEHAVIOR OF CORROSION-PROTECTED CABLE BOLTS

7.1 The aim of the laboratory axial tests

The aim of the axial tests was to determine the effect of the corrosion protection on axial load-displacement behavior of the steel strand with different embedment lengths.

7.2 The test apparatus and the test procedure

The laboratory double pipe axial tests of test bolts were conducted by grouting the test bolts inside steel pipes, which provided very practical, comparable and inexpensive test system. The same principle of double pipe test system has been used widely around the world in testing rock bolts and cable bolts.

The test apparatus was modified from the basic universal test system and consisted of a pulling device (hollow ram hydraulic jack), electric hydraulic pump, RHS-test frame, wedge system for the connection of test steel pipes, measuring instruments and a portable PC (Figure 5).

Three inductive displacement transducers measured the displacements of the test bolts. The displacements were measured not only from the middle (D1), the most important place, but also from both ends of the bolt (D2 & D3) where the displacements between

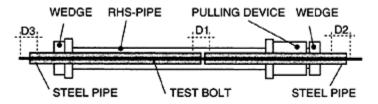


Figure 5. The double pipe test system (Satola & Hakala 2001).

the end of the steel pipe and test bolt were measured. The measuring data from transducers D2 and D3 were used to determine the general bond failure in every test.

After placing the test sample in the right position in the double pipe test machine, the test began, and increasing the pressure from the hydraulic pump, the load steadily rose at a rate of 10 kN/min. The test bolt was loaded until the failure of the test bolt occurred or the stroke was completed.

The most of the test bolts were tested with the embedment length of 0.25, 0.5, 0.75, 1.0 and 2.0. Cyclic axial tests were performed to determine the onset of non-elastic behavior and axial tests of preetched test bolts were carried out to determine the effect of corrosion on capacity. All tests and their results are fully reported in the references (Satola & Hakala 2001, Satola et al. in prep.).

7.3 Test results and discussion

The corrosion protection treatment on the surface of the test strand improved the bond strength between the test bolt and grout resulting in a higher stiffness of the behavior of the bolt. This was clearly seen from the shapes of the curves of the epoxy-coated steel strands and the galvanized steel strands with every tested embedment length: 2.0, 1.0, 0.75, 0.5 and 0.25 meters (Figures (6–10).

The bulbed strand had a high stiffness because of its modified geometry. The bulbs in the strand worked as anchors increasing the bond strength. The bulbed strand has two bulbs a meter.

With the embedment length of 1 meter some of the test bolts of galvanized steel strands went broken, but not all of them. With the embedment length less than

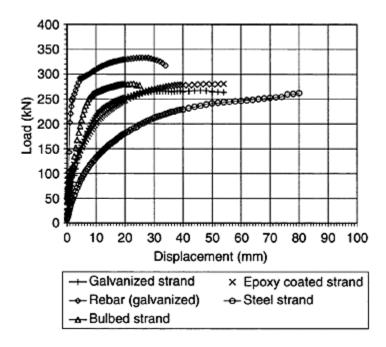


Figure 6. Axial double pipe test results. The average load-displacement curves of each test bolt type with the embedment length of 2000 mm.

1 meter the slippage took place instead of failure of strand. The same behavior occurred with epoxy coated steel strands indicating that the critical embedment length of both the galvanized steel strand and epoxy-coated steel strand was around 1 m. The corresponding value for the conventional cable was almost 2 meters. Conventional cables were tested only with the embedment length of 1 and 2 meters.

The epoxy coated steel strands had the highest ultimate bond strength (the ultimate load divided by the bond area) of the all steel strand types with every tested embedment length. Rebar has the highest ultimate bond strength of all the test bolts (Figure 11).

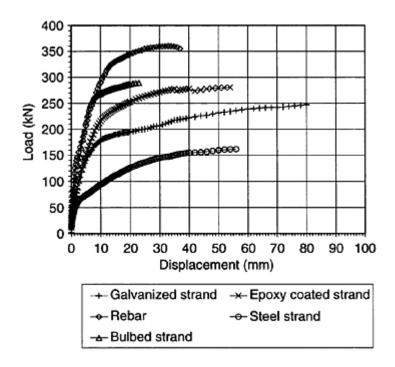


Figure 7. Axial double pipe test results. The average loaddisplacement curves of each test bolt type with the embedment length of 1000 mm.

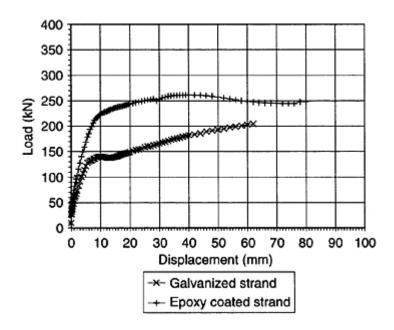


Figure 8. Axial double pipe test results. The average loaddisplacement curves of each test bolt type with the embedment length of 750 mm.

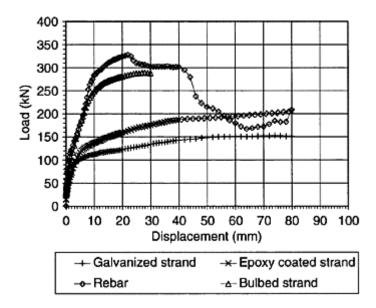


Figure 9. Axial double pipe test results. The average loaddisplacement curves of each test bolt type with the embedment length of 500 mm.

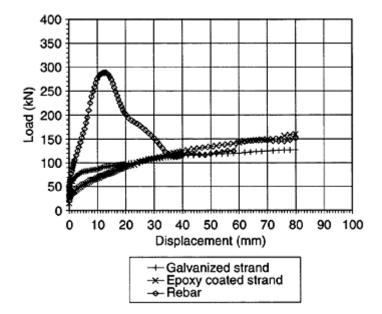


Figure 10. Axial double pipe test results. The average load-displacement curves of each test bolt type with the embedment length of 250 mm.

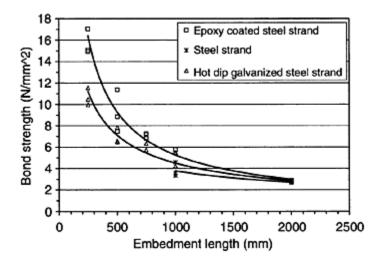


Figure 11. The ultimate bond strength of conventional cable bolts and corrosion-protected cable bolts.

8 CONCLUSION

To summarize the applicability of each strand type according to the sensitivity to corrosion, the following guidelines can be given:

Plain strand

- Corrosion rate depends on the stress level and environment. General corrosion rate of plain strand can be 25% higher than that of rebar.
- The stress level can increase the general corrosion rate up to two times higher compared to that of unloaded strand.
- Low pH and high chloride concentration can cause localized corrosion. Both longitudinal and transverse grooves can appear reducing the load bearing area.
- Crevice corrosion between the wires was not detected.

Epoxy-coated steel strand

- The use of epoxy coating gives longer service time. Corrosion will begin when ground water has penetrated the coating.
- Protection time depends on coating thickness and environment. Low pH and/or high concentration of dissolved salts give shorter times.
- Corrosion rate under the coating is comparable to that of uncoated strand.

Galvanized (hot dip galvanized) steel strand

 The zinc coating gives additional operating time; corrosion of steel strand will begin only after zinc coating has been dissolved. - The general corrosion rate of galvanized strand is the same as that of galvanized rebar.

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Corrosion assessment of ground support systems

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ABSTRACT: A Corrosion Assessment System has been developed to systematically measure and record various parameters associated with the ground water and atmosphere in mines. The measurements are complemented by qualitative and quantitative assessments of the condition of reinforcement and support systems relative to their time of installation. These measurements and assessments are recorded. Data have been collected at eight mine sites throughout Australia. These sites exhibit a wide range of ground water qualities. A correspondingly wide range of observed degrees of corrosion for similar ground control systems has been established.

1 INTRODUCTION

The corrosion of rock reinforcement and support systems and the effect on their load bearing capacities has not been widely researched and is generally not well understood. While there is much literature relating to the phenomenon of corrosion it is not always applicable to the hard rock underground mining environment.

The purpose of rock support and reinforcement is to maintain excavations safe and open for their intended lifespan. The types of support and reinforcement required in a particular location are dependent upon several factors that include the strength of the rock mass, the geometry of the excavation, the stresses present in the rock, the blasting practices, and the weathering and corrosion processes applicable at the site (Villaescusa 1999, Windsor & Thompson 1993). Sundholm (1987) suggests that corrosion is one of the major factors determining which reinforcement type can be used as permanent support.

Corrosion of reinforcement is a poorly understood mechanism and no systematic field study has ever been undertaken within the underground mining industry in Australia. The WA School of Mines is currently conducting a 3-year research project into corrosion in underground mining. This study aims to determine the main corrosion mechanisms affecting rock reinforcement elements used underground and to assess the effectiveness of existing corrosivity classifications.

2 REVIEW OF EXISTING CORROSIVITY CLASSIFICATION SYSTEMS

A wide variety of corrosivity classifications are available in publications; however most are specific to certain environments and conditions. The following is a review of the main classifications that may be useful in the assessment of ground control systems in underground mines.

2.1 Soil corrosion classification

The probability of corrosion of metals in soil has been widely researched with numerous classifications. Parameters that are usually examined include soil type, resistivity, water content, pH, buffer capacity, sulphides, neutral salts and sulphates, presence of groundwater, the horizontal and vertical homogeneity (i.e. difference in soil structure), and the electrochemical potential (http.njuct.edu.cn/MatWeb/soil/table603.htm).

The use of soil corrosivity classification for the underground hard rock environment is not recommended because of the obvious different environments in which the corrosion takes place.

2.2 Atmospheric corrosion classification—ISO

The ISO 9223:1992 standard classifies the corrosivity of an atmosphere based on measurements of time of wetness (TOW), and pollution categories, sulphur dioxide (SO₂) and airborne chlorides. The corrosivity of the atmosphere is divided into five categories ranging from

	U			U			
Mine site	TDS (mg/l)	(pH)	Cl(mg/l)	SO ₄ ²⁻ (mg/l)	Ca ²⁺ (mg/l)	DIN W ₁	Value W_L
				(119/1)			
1	230,000	5.80	180,000	24,000	310	-24	-14
2	5,400	8.12	1,570	1,287	187	-11	-5
3	13,000	8.80	7,700	170	430	-17	-7
4	44,000	7.40	23,000	6,000	910	-12	-7
5	120,000	6.40	69,000	7,800	1,600	-17	-8
6	6,100	7.70	1,500	800	300	-11	-5
7	28,000	5.90	16,000	1,500	2,700	-15	-23

Table 1. DIN corrosion classification values for eight Australian mine groundwaters.

8	160,000	7.00	93,000	3,100	4000	-21	-6

very low to very high with corresponding corrosion rates for carbon steel and zinc.

The TOW is a key parameter as it determines the amount of time the electrolyte layer is present on the surface of the metal. For this standard it has been defined as the time period during which the relative humidity is in excess of 80% and the temperature is above 0°C. This standard is widely used to classify atmospheric corrosion potential in many different environments; however, it often needs to be modified and calibrated to that specific environment.

2.3 Groundwater corrosion classification—DIN

The German DIN 50929 classification assesses the corrosive potential of water based on

the flow (if the location is submerged), the chloride (Cl⁻) and sulphate $(SO_4^{2-})_{content}$, acidity, calcium ion (Ca²⁺) content, and pH. Each parameter is given a positive or negative numerical rating based on the effect they have on the corrosivity and then summed to obtain the probability of corrosion. The more negative the number, the more corrosive the water. The classification is separated into four corrosivity levels with the most corrosive group having a W₁ less than -8.

Table 1 shows the DIN values for various mine groundwaters collected during this project within Australia. W_1 values are for steel, while W_L values are for zinc. All the mine groundwaters have W_1 values less than -8 and are placed in the highly corrosive category.

The indications are that the DIN classification system does not distinguish between widely different mine waters, especially for the high total dissolved solids (TDS) groundwater generally found in Australian underground mines.

2.4 Underground hard rock corrosion classification—Li & Lindblad (1999)

Li & Lindblad (1999) have proposed two corrosivity classifications for the underground environment with relation to the corrosion of steel rock bolts. The first

	510		
Mine site	W _{wet}	Corrosion description	Corrosion rate (mm/yr)
1	8.6	Severe	0.15-0.30
2	6.8	Severe	0.15-0.30
3	12.4	Very severe	>0.30
4	11.9	Very severe	>0.30
5	10.6	Very severe	>0.30

Table 2. Li & Lindblad (1999) corrosion classification ratings for eight Australian mine groundwaters.

6	18.6 Very severe	>0.30	
7	16.4 Very severe	>0.30	
8	11.2 Very severe	>0.30	

classification is for wet rock conditions and the corrosion-related parameters used are pH, dissolved oxygen, resistivity, ambient temperature, rock mass quality and precipitation of calcium carbonate. The second classification system is for dry rock conditions using the following parameters; deposition rate of sulphur, nitrogen oxides and chloride, as well as the relative humidity and ambient temperature.

Table 2 shows the Li & Lindblad (1999) classification for wet rock conditions using the same Australian mines analysed in Table 1. The majority of the mine groundwater is rated (W_{wet}) in the very severe corrosion category, with a large weighting being given to temperature. This classification has been developed for European groundwater and appears to place too much emphasis on factors that are not as relevant to Australian groundwaters.

2.5 Saturation indices

Water saturation indices relate the solubility of dissolved ions to their tendency to precipitate. One of the most notable is the Langelier Saturation Index (LSI), which is an indicator of the degree of saturation of water with respect to calcium carbonate. The LSI is defined as the difference between the measured pH and the pH_s at saturation in calcite or calcium carbonate. If LSI is negative, then there is no potential to scale, as the water will dissolve the calcium carbonate. If the LSI is positive, then scale can form and calcium carbonate precipitation may occur protecting the metal from corrosion attack. The majority of mine groundwaters showed the potential for scale to form (Table 3). However, the general consensus is that these types of indices are indicators of the formation of scale and not of corrosivity and they do not include other important corrosivity parameters such as dissolved oxygen, temperature, dissolved ions and water velocity.

2.6 Assessment of current corrosivity classiflcations

The use of soil corrosivity classifications in the hard rock underground environment is not feasible due to

Mine N°	LSI	Potential to scale
1	-2.2	2 No potential
2	0.2	1 Precipitation may occur
3	0.4	3 Precipitation may occur

Table 3. LSI ratings for eight Australian mine groundwaters.

4	0.62 Precipitation may occur
5	-0.77 No potential
6	-0.14 Borderline scale potential
7	-1.11 No potential
8	-0.01 Borderline scale potential

the different environments in which the corrosion takes place.

The ISO 9223 and the dry corrosion classification of Li & Lindblad (1999) have similar input parameters that appear to work reasonably well for determining the atmospheric corrosion environment in underground mines. A difficulty arises however when determining atmospheric variables that are constantly changing as mining progresses.

The most destructive and complicated corrosion environment to predict is that related to the influence of groundwater. Both the DIN and Li & Lindblad (1999) classifications examine the corrosive potential of water and generally rate Australian mine groundwater as highly corrosive. These classifications have been developed in Europe where different groundwater conditions from Australia exist. Australian groundwaters, especially in the southern Yilgarn Craton, have high temperatures and extremely high salinity. The mildest groundwater collected in Australia rates in the worst category for the DIN classification and the second worst for the Li & Lindblad (1999) classification. It is clear that these classifications were not developed for groundwaters with high TDS and temperatures, as found in many Australian underground mines.

3 CORROSION ASSESSMENT SYSTEM

A major objective of a Corrosion Assessment System is to collect data in a systematic way that will enable observed corrosion of reinforcement and support systems to be related to the various conditions existing within the underground environment. It should also provide an assessment of the current condition of the reinforcement and support systems in various areas of the mine.

3.1 Corrosion potential of a rock mass

All corrosion of support and reinforcement in underground mines is due to contact with aqueous solutions. The actual rate and form of corrosion is controlled by the different environments to which ground support is subjected. Because environmental conditions in underground mines are never homogenous and are constantly changing, only approximations can be made for classifying the environment and such approximations must be constantly reviewed.

Essentially, corrosion in an underground environment can be classified into two controlling environments. Atmospheric corrosion, which can be defined as the corrosion of materials exposed to air and its pollutants. The second type of environment is the combination of groundwater and the atmosphere. Corrosion due to the presence of groundwater is significantly more aggressive than atmospheric corrosion and only occurs when the reinforcement and support are in direct contact with groundwater. The corrosion itself may be due to a combination of groundwater and the atmosphere variables; however when groundwater is present it is usually the controlling factor to such an extent that atmospheric variables are rarely considered.

3.1.1 A tmospheric variables

The majority of reinforcement and support in underground mines is affected by corrosion from attmos-pheric variables. The primary variable is the time of wetness (TOW), which is a function of temperature, relative humidity and the surface shape of the metal. The TOW refers to the period of time during which the atmospheric conditions are favourable for the formation of a surface layer of moisture (electrolyte) on a metal, which is integral to the corrosion process. The level of atmospheric contaminants often controls the rate of atmospheric corrosion. Pollutants such as sulphur dioxide (SO₂) mix with the electrolyte producing sulphuric acid. Other primary contaminants include atmospheric chlorides, nitrogen compounds and dust particles. The process of blasting and the use of diesel equipment are the major sources of these contaminants, which are further concentrated due to the restricted ventilation system, especially at return airways.

3.1.2 Groundwater variables

Groundwater flowing from the rock mass is present in most underground mines, however it interacts with only a small percentage of reinforcement and support systems. The elements are generally not submerged, but are located in a water/atmosphere interface.

Groundwater variables that influence the corrosivity of waters include:

- pH-the control the pH value exerts on the corrosivity of a system depends to a large degree on the solubility of the corrosion product (usually the oxide) formed on the metal surface. Under normal conditions, the corrosion rate of steel is independent of pH values between 4.5 and 10.
- Dissolved Oxygen—dissolved oxygen is probably one of the most important factors that influence metal corrosion in a mine water system. It significance lies in the fact that it is the most common cathodic reactant present in natural waters.

$$O_2+2H_2O+4e^-\rightarrow 4OH$$

The concentration of oxygen in seawater at ambient temperatures is approximately 8 mg/l. Dissolved salts and temperature often control the oxygen concentration.

- Water Velocity—as velocity causes a mass flow of oxygen to the surface of the metal, and corrosion is dependent on oxygen concentration, the corrosion rate will increase with an increase in water velocity. Also high water velocity can cause erosion of corrosion products decreasing their protective nature.
- Chlorine and Sulphates—aggressive anions in the water such as chlorides and sulphates increase the corrosion rate by lowering the electrical resistivity of corrosion cells and

playing a significant role in the penetration and breakdown of any protective film that might have formed on the metal surface.

 Scaling—precipitation of salts such as carbonates may produce a film or scale that slows or prevents corrosion. Calcareous deposits are sometimes formed at the cathodic sites. However, the high total dissolved salts encountered with mine groundwater greatly impedes the ability of corrosion reducing carbonate scaling by forming different chemical deposits that are porous and less effective as a protective coating.

3.1.3 Rock mass quality variables

The rock mass quality indirectly influences the corrosive potential of a mine environment. The rock mass structures are primarily important as they provide a conduit for ground and fill water flow. Major geological structures such as faults and shears allow the flow of groundwater into the mine from surrounding aquifers. The extent of the area affected by the water is increased by the presence of interconnected, dilated joint sets, which allow the ground water to travel and dissipate at significant distances from the source. Opening of the joints can occur during the initial excavation or subsequently at a later date due to blasting or stress changes.

The mineralogy associated with the different rock masses is not expected to influence the corrosion potential of an environment. The minerals are generally inert and do not enhance the corrosion processes in any major way. An exception to this rule is for sulphide minerals. These reactive minerals oxidise, creating very localised acidic conditions as well as possibly creating an electrochemical corrosion cell with the rock reinforcement. This may accelerate corrosion, however, such occurrences are considered to be localised.

3.2 Condition of reinforcement and support

The conditions of reinforcement and support systems are dependent upon their age, the corrosive potential of the environment and the types of reinforcement and support used. It is often difficult to assess the condition of reinforcement, especially for fully grouted and resin encapsulated elements. During this research project, observing the internal condition of ungrouted friction rock stabilisers with the use of a borehole camera has had some success, but only the internal section of the bolt can be observed.

Assessing the condition of the surface support and extrapolating to include the condition of the reinforcement is not recommended. The excavation surface and the internal rock mass are two separate envir- onments with different rates and forms of corrosion. An example is given in Figure 1.

Currently as part of this research project an overcoring drill rig is being developed at the Western Australian School of Mines (WASM) to overcore

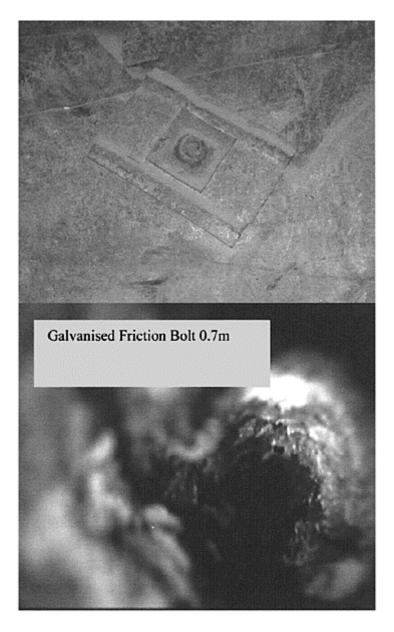


Figure 1. Galvanised friction bolt showing no corrosion of plate and severe internal corrosion located about 0.7 m from the collar. reinforcement elements in-situ. This will provide detailed information on how, where and what type of corrosion is occurring on reinforcement elements, as well as the effectiveness of encapsulation by resin and cement grouts.

3.3 Mechanisms of corrosion

The nature of the corrosion process will depend on the interaction between a material and its environment. When specific conditions in the material and the environment occur characteristic forms of corrosion result. Recent data collection at several Australian underground mines have identified three main forms of corrosion, which are supported in part by the work reported by Ranasooriya et al. (1995) and Gamboa & Atrens (2003). The main types of corrosion observed on reinforcement and support are uniform or general corrosion, localised corrosion in the form of pitting, and stress corrosion cracking. These are not the sole types of corrosion observed in the underground environment, but are considered the most common and detrimental.

3.3.1 Uniform or general corrosion attack

Uniform or general corrosion occurs when the anodic and cathodic areas on the metal surface change position continuously. As a result, the corrosion that occurs at the anodic areas is spread evenly over the surface of the metal. Other common forms of corrosion occur when this exchange of anodes and cathodes is prevented in some way, so that the corrosion becomes localised. Commonly uniform corrosion occurs when most metals are exposed to the atmosphere or submerged in water. The rate of corrosion often declines with time due to the build up of corrosion products on the surface of the metal (CISA 1994).

3.3.2 Localised pitting corrosion

Pitting corrosion is the highly selective attack of passive metals at defects in the passive oxide layer. The corrosion attack is in the form of pits, usually covered by the corrosion products or remnants of the original protective layer. Pitting is considered to be more dangerous than uniform corrosion because it is difficult to detect, predict and design against (Roberge 2000).

Pitting usually occurs in solutions containing chlorides, and becomes autocatalytic (i.e. it stimulates itself once initiated). This occurs when a charge imbalance occurs as the anodic reaction becomes localised within the pit, and as cations are liberated into the pit solution. As a result, anions such as chloride ions diffuse from the bulk solution to the pit to equalise the charge. The presence of chlorides further stimulates corrosion by forming soluble corrosion products (CISA 1994) as well as acidification. The depth of penetration of pitting can be significantly higher than the rate of uniform corrosion.

Pitting is the most common form of corrosion of steel in concrete and work by Ranasooriya et al. (1995) identified pitting corrosion as the most common of all types of corrosion on collected samples of friction rock stabilisers.

3.3.3 Stress corrosion cracking (SCC)

Stress corrosion cracking (SCC) is used to describe service failures in materials that occur by slow environmentally induced crack propagation (Jones & Ricker 1990). The cracking is induced from the combined influences of tensile stress and a corrosive environment and crack propagation proceeds slowly until it reaches its critical length and rapid fracturing occurs suddenly.

The required tensile stresses may be in the form of directly applied stresses or in the form of residual stresses. Usually, most of the surface remains unattacked, with fine cracks penetrating into the material and thus is very difficult to detect and damage is not easily predicted. (www.corrosion-doctors.org/Forms/ scc.htm). Crosky et al. (2002) have found that steel with low fracture toughness is particularly susceptible to this type problem.

Recent experimental results from Gamboa & Atrens (2003) on SCC of rock bolts indicates that SCC begins when sheared by moving rock strata and that the critical crack length can be of the order of only a few millimetres for rock bolts. However, laboratory testing indicates that SCC occurs in low pH waters unlike most Australian mine groundwaters, which have an almost neutral pH. However, such pH may be reached locally due to the presence of bacteria or sulphides or by previous leaching mining methods contaminating the groundwater.

3.4 Data collection sheet

To simplify the collection of data and to ensure that all possible variables affecting corrosion were identified, a Corrosion Assessment System was developed that systematically identified the following information for collection at mine sites:

Site Specification

- Excavation type and age

- Rock Mass
 - Intact Rock
 - Geological Discontinuities
- Environmental Factors
 - Groundwater
 - Atmosphere
 - Stresses
- Ground Control Scheme
 - Reinforcement Systems
 - Support Systems
 - Other Component Systems

The rock mass and environmental factors will characterise the potential of a particular underground location to cause corrosion. Observations of actual corrosion on installed reinforcement and support systems can be used to identify the corrosion resistance of different systems. The corrosion data collection sheet is shown in its entirety in the Appendix with rock mass condition diagrams taken from Ortlepp (1992) and Heslop (1997).

4 CASE STUDIES

Using the corrosion assessment system described here a number of Australian underground mines have been inspected to determine the corrosive potential of the underground environment and the condition of reinforcement and support in those environments.

Based on this work two kinds of corrosive environments have been identified. Those affected by only the mine atmosphere and those affected by both the mine atmosphere and mine groundwater flowing from the rock mass.

4.1 Mine A tmosphere

As indicated previously, atmospheric corrosion can be defined as the corrosion of materials exposed to air and its pollutants. It was found that mine atmospheres do vary considerably with respect to moisture content, temperature, time of wetness and contaminants throughout the mine. Over time as mining progresses and the ventilation circuit changes, these parameters change.

Table 4 shows the range over which temperature and relative humidity is present at various mine sites. Generally for the Western Australian mines the higher temperatures and relative humidity occur in return airways, ore drives and where the air velocity is low. Declines and larger crosscuts commonly have the lowest temperature and humidity, which makes them less favourable for atmospheric corrosion attack. However two mines investigated (Mine sites 6 & 7) have high temperatures and humidity due to higher rock and water temperatures and the common use of

Mine site	Temperature (°C)	Relative humidity (%)	
1	18–24	50–70	
2	15–23	65–90	
3	17–30	50–79	
5	21–26	75–90	
6	25–32	80–95	
7	30–45	80–95	
8	20–29	60–90	

Table 4. Temperature and relative humidity range of eight Australian underground mines.

secondary ventilation. These atmospheres are moderately corrosive and for such mines the risk of atmospheric corrosion is higher.

Fortunately, the rate of atmospheric corrosion is generally uniform and relatively slow with corrosion rates for mild steel generally less the 0.05 mm/yr.

However, ground support within long-term excavations can become severely corroded over extended periods of time. Atmospheric corrosion is usually not a problem for shortterm excavations.

4.1.1 Condition of reinforcement and support

Reinforcement and support under attack from atmospheric corrosion were generally in fairly good condition. This is attributed to the low temperatures and relative humidity found throughout most mines, the relatively short life expectancy of the ground control in respect to the corrosion rate and the extensive use of galvanised and fully encapsulated reinforcement for long term control.

It is expected that few problems will occur from atmospheric corrosion at most mine sites. It has been seen on numerous occasions that black friction bolts form only uniform surface corrosion after two years of installation in a mild atmosphere (Fig. 2). However, some mines, for various reasons already discussed, have moderately corrosive atmospheres. When combined with a long mine life expectancy, corrosion from atmospheric variables can cause significant problems as illustrated in Figure 3. The ungrouted point anchor bolt is in excess of 15 years of age and has had severe localised corrosion along the element.

4.2 Mine groundwater

Corrosion attack due to mine groundwater affects a minority of ground support within a mine. However, the attack can be highly aggressive and caused by a number of corrosion mechanisms with damage difficult to predict. Mine groundwaters are generally encountered in the presence of major structures and near draw points of stopes that have been backfilled. The majority of groundwater encountered by underground mining activities in the Yilgarn occurs from hard or fractured rock aquifers in igneous and metamorphic rocks and more than one aquifer may be controlling the flow of groundwater. The rock itself is generally impermeable, but fractures, joints and weathering allow a degree of porosity and permeability.

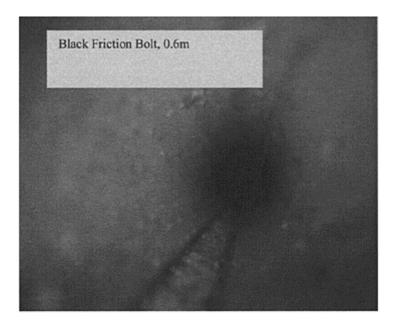


Figure 2. Black friction bolt after two years of installation in 70–80% humidity atmosphere.

Groundwater flow from geological structures is rarely constant and regular over an extended time period. The reasons may be environmental and related to depletion of an aquifer, as well as insufficient recharge or man-made changes such as nearby stoping diverting the flow of water. Excess water from cemented backfill also flows into a rock mass and while not technically groundwater, its influence on surrounding ground support is similar. Areas affected by backfill water are usually restricted to drawpoint levels and often for limited amounts of time.

At depth, groundwater becomes less common as there is little weathering and high in-situ ground stresses usually keep the structures tight, so that ground water flow is severely restricted. Subsequent mining activity redistributes the stress field and structures open up allowing groundwater to flow into previously dry areas. However, experience has shown the rate and occurrence of groundwater flow generally decreases with depth for all fractured rock aquifers.

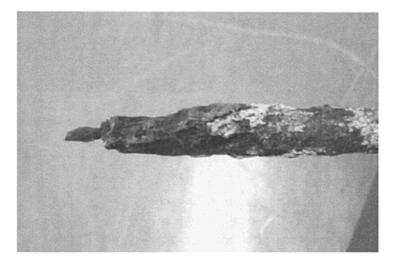


Figure 3. Necking of ungrouted point anchor bolt due to atmospheric corrosion.

The presence of water is not always obviously visible on the surface of excavations but may still be present deeper in the rock mass, where corrosion of reinforcement may occur. These areas are classified as damp and are often revealed by white salt crystal build up at the surface of excavations and a damp appearance within the rock surface.

Mine groundwater samples were collected from eight Australian underground mines and were tested in-situ for total dissolved solids (TDS), pH, dissolved oxygen (DO₂) and temperature. The samples were also subsequently analysed for Na⁺, K⁺, Ca²⁺, Mg²⁺, C1⁻, CO_3^{2+} , HCO_3^{-} , SO_4^{2-} and NO_3^{+} ions. A summary of the main constituents associated with corrosion is shown in Table 5.

The testing and analyses of the groundwaters showed a TDS range of 3,700 ppm to 230,000 ppm with the major ions present being Na⁺, Ca²⁺, Cl- and **SO₄²⁻**. In terms of corrosiveness the Cl⁻ and **SO₄²⁻** ions are the most important and generally the concentrations were very high. The dissolved oxygen ranged from 0.73-4.1 mg/1 and is directly related to the TDS and temperature of the water. The pH ranged from 5.80-8.12 indicating that the groundwaters are relatively neutral and at this level pH no longer plays a direct role in corrosion although it can still affect the solubility and equilibrium of other ions. The Ca²⁺ and **HCO₃⁺** ions are only minor constituents of the groundwater. Compared with the levels of chlorides and sulphates, it is not believed they provide any corrosion protection.

4.2.1 Condition of reinforcement and support

In the case of fully encapsulated reinforcement, dilation of geological discontinuities is likely to create cracks within the cement grout or resin column leading to corrosion (Fig. 4). This may leave the elements

Mine N°	TDS (ppm)	DO ₂ (mg/1)	Temp (°C)	рН	Ca ²⁺ (mg/1)	HCO ₃ (mg/l)	Cl ⁻ (mg/1)	SO₄²⁻ (mg/l)
1	230,000	0.73	26	5.80	310	50	180,000	24,000
2	5,400	3.54	26	8.12	187	73	1,570	1,278
3	13,000	3.92	27	7.67	430	10	7,700	170
4	44,000	1.13	26	7.40	910	250	23,000	6,000
5	120,000	3.23	22	6.40	1,600	75	69,000	7,800
6	3,700	4.1	30	7.70	300	48	1,500	800
7	28,000	4.13	33	5.90	2,700	35	16,000	1,500
8	160,000	3.2	28	7.00	4,000	35	93,000	3,100

Table 5. Groundwater characteristics of eight Australian underground mine groundwaters.

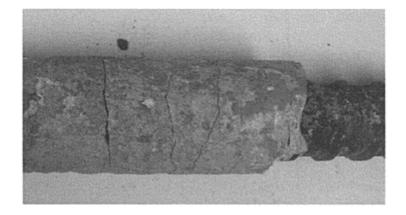


Figure 4. Cracking of a resin grout column exposing the black rebar.



Figure 5. Overcored resin encapsulated rock bolt using the WASM bolt overcoring rig.

exposed at the areas where the groundwater is more likely to flow. The timing of such occurrences is difficult to predict, but stress changes and blast damage may promote dilation of rock mass discontinuities.

In the case of cement grouted elements, the cement itself is a corrosion inhibitor due to its high alkalinity and is also self-healing, as the grout can reform in the crack protecting the element. Results from laboratory testing suggest that a minimum crack width of greater than 1-2 mm is needed before self-healing cannot occur.

Research into corrosion along the axis of fully encapsulated bolts has not been completed, as trials of bolt overcoring (Fig. 5) have just started at the WA School of Mines. Nevertheless, numerous friction bolts have been examined internally to gauge their level of corrosion in relation to their environment.

Friction bolts are highly susceptible to corrosion attack because of their thin steel thickness and open design of the bolt. Figure 6 shows the internal condition

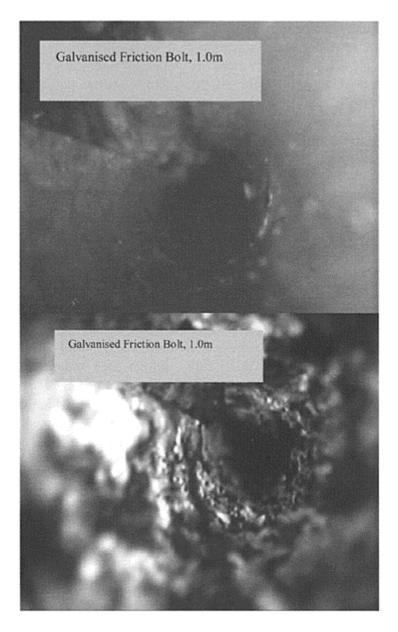


Figure 6. The internal condition of a galvanised friction bolt after 2 months (top) and 4 months (below) in an environment strongly affected by groundwater (same mine site).

of two galvanised friction bolts of different ages. The upper bolt is shown after two months of installation and the bottom after four months of installation in a wet shear zone with water at mine 6. A considerable increase in the corrosion rate from essentially non-corroded to strong corrosion was observed in the interval of two months. As a general rule, corrosion on galvanised friction bolts is not experienced before 2–3 months regardless of the environment. However, once the protective zinc coating has been compromised, corrosion of the steel may follow very quickly. The corrosion rate actually decreases over time due to the protection provided by the corrosion products. The corrosion products are a mixture of iron rust and deposited salts. These products provide some protection from uniform corrosion but at the same time promote pitting corrosion.

The susceptibility of weld mesh to corrosion appears to be moderate. Strongly corroded mesh samples have been laboratory tested to determine their failure loads, which range from 0-40% less than failure loads determined from non-corroded samples.

4.3 Summary

Two corrosive environments have been established for the underground environment; those affected by atmospheric variables and those affected by groundwater variables. Often a clear distinction between the two environments cannot be established. Mine atmospheres range from mildly to moderately corrosive and generally do not cause a problem for short-term support. Groundwater corrosion is considerably more aggressive and is a problem for both short and long term ground control, especially for thin walled reinforcement such as friction bolts. While cement grout and resin encapsulation offer some protection from the corrosive environment, dilation of joints causes cracking of the protective barrier so the element is at risk. It has been found that galvanising of ground control elements helps protect the underlying steel from corrosive attack by natural groundwaters.

5 LABORATORY TESTING

5.1 Wheel tests

A number of laboratory-based experiments were undertaken in conjunction with the WA Corrosion Research Group at Curtin University using the accelerated corrosion wheel test methods. In this test, a metal coupon is placed within a glass bottle containing the water sample. The bottle is capped, placed on a wheel inside an oven and rotated for the required period (generally 6 hours). The test provides stirred conditions at a controlled temperature. The test is excellent for screening the performance of a large number of metals, in different waters, under the same condition. It is important to appreciate that the test vessel is a closed system and the chemistry inside the vessel will change according to the corrosion process. Two series of testing has been completed using synthetic and actual mine groundwaters.

5.1.1 Synthetic groundwaters

Five synthetic mine groundwater types were prepared based on data from Australian underground mine groundwaters. They ranged from dilute (Cl⁻ 500 ppm, SO_4^{2-} 50 ppm) to extreme (Cl⁻ 12,500 ppm, SO_4^{2-} 12,500 ppm) with a pH of 7 at 25°C and were tested on four common reinforcement elements. These were black thread bar, black friction bolt, black strand cable and galvanised strand cable. The corrosion rates for each synthetic water and each reinforcement element are shown in Figure 7. The results indicate that there is a steady decrease in the corrosion rate of the black

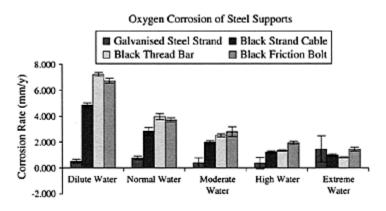


Figure 7. Corrosion rates of various reinforcement elements in a range of synthetic mine groundwaters.

Table 6. Corrosion rates of mild steel in actual mine groundwaters.

Mine site	1	2	3	4	5
Corrosion rate (mm/yr)	1.35	2.57	7.34	3.65	4.24

reinforcement as the water quality become worse. This is attributed to a lowering of the dissolved oxygen content due to the higher TDS and an increase in scaling that prevents oxygen diffusion to the metal surface. The galvanised strand gave much lower corrosion rates and, if anything, the corrosion rate increased with salt concentration. The precision of these measurements was quite poor due to the non-uniform surface of the galvanised samples.

5.1.2 Mine groundwaters

A second set of tests was conducted using mine groundwater collected directly from underground rock masses. The corrosion rate was tested on mild steel and the corrosion rates ranged from 2–7 mm/yr (Table 6.). This rate is quite extreme and overstates the actual rate of corrosion. The predicted rate of corrosion by the oxygen wheel test is assumed to be the initial rate of corrosion. As the process continues the corrosion products actually slow the rate of corrosion. This statement is supported by field evidence, which did not detect the predicted rate of corrosion reinforcement on any environment underground.

When comparing the results with the groundwater analysis it is concluded that one groundwater parameter does not control the rate of corrosion.

6 OUTCOMES FROM CORROSION CLASSIFICATIONS AND LABORATORY TESTING

Comparing the existing corrosion classification rankings to the actual corrosion rate measured from the

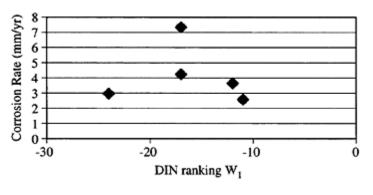


Figure 8. Relationship between DIN ranking W_1 and wheel test corrosion rates.

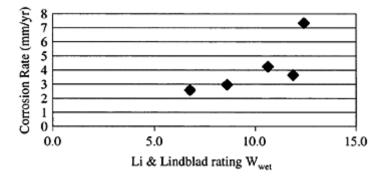


Figure 9. Relationship between Li & Lindblad ranking and wheel test corrosion rates.

oxygen wheel test the relevance of each classification for Australian mine groundwaters is appreciated. The DIN classification (Fig. 8) shows no trend. With this method the corrosion rate is expected to increase as the rating decreases.

The Li & Lindblad (1999) classification shows an increase in ranking for the increasing corrosiveness of the water (Fig. 9) as predicted by the classification.

No correlation is seen when comparing the corrosion rate to the Langelier Saturation Index rating (Fig. 10). With this method those waters having a positive rating should have a lower corrosion rate if protective calcium carbonate scaling is forming.

It is accepted that the sample volume is small, although research is progressing and early indications are that the classification that best predicts the corrosiveness of the waters is that proposed by Li & Lindblad (1999).

7 CONCLUDING REMARKS

A corrosivity assessment system has been developed and tested on a number of underground environments. The method characterises the rock mass, environmental and ground control schemes contributing or affected by corrosion. The parameters collected have been used to evaluate existing corrosivity classifications systems,

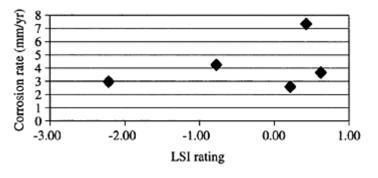


Figure 10. Relationship between the Langelier Saturation Index and wheel test corrosion rates.

which are generally not appropriate to the Australian mining conditions. The corrosivity classification by Li & Lindblad (1999) appears to be the most applicable.

ACKNOWLEDGEMENTS

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				s	ITE SPE	CIFIC	ATION					
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APPENDIX

			STR	ESSES				
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Ground support in mining and underground construction 1014

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System Number	Date of Installation	Component	Type	Dimensions	Material	Coating	Corrosion Description	Photo
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KI I		External Fixture						
		Plate						
		Element						
R2		Internal Fixture						
RE		External Fixture						
		Plate						
		Element						
R3		Internal Fixture						
~ [External Fixture						
		Plate						
		Element						
R4		Internal Fixture						
~		External Fixture						
		Plate						
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System Number	Date of Installation	Component	Турс	Dimensions	Material	Coating	Corrosion Description	Phote
S1		Plate						
S2		Strap						
\$3		Mesh						-



11 Surface support

Hydro scaling and in-cycle shotcrete at Waroonga mine, Western Australia

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> J.Mitchell Jetcrete Australia Pty Ltd

B.Upton Byrnecut Mining Pty Ltd

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ABSTRACT: In-cycle shotcrete (ICS) is used in several Australian mines for developing through extreme ground condition. Trials at Agnew Gold Mining Company's Waroonga mine have investigated the potential for ICS together with hydro scaling to replace conventional jumbo scaling and mesh and bolting with ICS as the primary method of ground control for development under non-extreme conditions. These trials have progressed from investigating the integration of the method into the mining cycle to scrutinising its productivity in an extended mine-wide trial, which may lead to the adoption of this technique on a permanent basis. The background to the use of hydro scaling, the objectives and results of the Agnew trials to date are discussed as well as the development of quality control/assurance criteria that may enable the full potential of shotcrete, as a support medium, to be exploited.

1 INTRODUCTION

1.1 Waroonga operations

The Waroonga site is located 370 km North of Kalgoorlie and 30 km West of Leinster, where the mine's predominantly fly-in-fly-out workforce is based. There has been sporadic mining and exploration activity at Waroonga for more than 100 years. The main periods of mining activity were:

- 1897–1911 Waroonga Gold Mines
- 1935-1948 East Murchison United Gold Mines

– 1986–1990 WMC Ltd Emu AG open pit

The current phase of operation began with the Waroonga open pit cutback of the Emu AG pit, by WMC in 2001. Gold Fields completed the cutback in early-2003, following acquisition of the Agnew and St Ives Gold Operations from WMC in December 2001. Underground mining at Waroonga resumed with portal establishment in early-2002 and initial decline development to access the Kim Lode, under the northern end of the pit.

Underground ore was first delivered from Kim Lode in late-2002 with development towards the Main Lode ore bodies, below the southern end of the pit and the old underground workings, commencing early the following year. There is potential to extend the current 3-year reserve at Waroonga to a 10-year plus mine life, with ongoing exploration drilling.

1.2 Hydro scaling

The use of high pressure water jetting for the scaling of fresh blasted rock has been the subject of trials in both mining and civil environments in Scandinavia and North America, with research having been undertaken at the Colorado School of Mines (CSM). The CSM work established optimum water jet pressures for hydro scaling and sought to improve adhesion across the shotcrete rock interface. It showed that a fourfold increase in bond strength could be achieved through the use of 3000–4000 psi (20.7–27.6 MPa) water jet pressures for scaling prior to shotcrete spraying. An overview of the encouraging results of this work is presented as this provided the confidence in hydro scaling as the key productivity driver for the trials at Agnew.

1.3 Waroonga ICS trials

The Waroonga trials, conducted by Jetcrete Australia and Byrnecut Mining, the principle underground contractor at Agnew, appear to have been the first attempt to integrate hydro scaling into the mining cycle. A strong motivation for the trials was the potential to minimise the use of production jumbos for mechanical scaling, that incurs excessive maintenance cost, whilst maximising their design purpose productivity.

An initial 11-day single heading trial of hydro scaling and ICS was conducted in the Main Lode exploration decline in March 2003 and was subject to time and motion studies. This short trial showed that the integration of the method into the mining cycle was possible, with benefit to drive profiles and ground support quality. However, more importantly the financial analysis indicated that if the potential productivity benefits were harnessed, then mine-wide implementation of this method could be cost-neutral compared to the more traditional method.

An extended three-month trial commenced in November 2003 using site-sourced aggregate and batch plant established on site. This trial was monitored closely and attempted to realise the productivity benefits and quantify some of the less tangible benefits of the method.

2 PREVIOUS HYDRO SCALING STUDIES

2.1 Kiruna mine underground trials

At LKAB's Kiruna sub-level cave iron ore mine in the late-1990s the support system in the production cross cuts consisted of grouted dowels and shotcrete. About 20,000 m³ of shotcrete was sprayed annually, mostly un-reinforced and to a nominal 40 mm thickness. Figure 1 shows the two types of shotcrete failure that were observed.

A water-jet scaling trial was conducted to compare shotcrete adhesion with this surface preparation technique to that from the normal boom mounted hydraulic scaling hammer with the rock surfaces cleaned by washing down with water at a pressure of 0.7MPa (100 psi), Malmgren & Svenson (1999). The prototype water-jet rig supplied water at a pressure of 20 MPa (2900 psi) and 0.2 m^3 /minute flow rate.

Table 1 shows that a threefold increase in shotcrete to rock bond strength (to 0.6 MPa) was achieved in

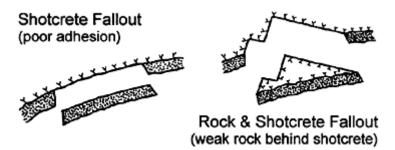


Figure 1. Shotcrete failure modes at Kiruna.

the Kiruna trials and there was notably less failure in rock behind the shotcrete interface. From this it was inferred that there was less scaling induced damage to the immediate wall rock in the trial section.

2.2 Colorado School of Mines investigations

The CSM research was initiated as part of a program to improve miner health and safety through the development of advanced technology for ground support and is documented by Kuchta (2001, 2002). The objective of the project was to develop equipment for scaling using high pressure water in order to:

- Remove miners from high risk areas and reduce their exposure to falls of ground; and

- improve the adhesion of shotcrete when applied as initial ground support.

Variable water pressures, ranging from 100–6000 psi (0.69–41.4 MPa), were used in the hydro scaling trial at the Army Tunnel in the CSM experimental mine. Six test panels on the tunnel sidewall in quartzfeldspar-(muscovite) gneiss and six concrete test panels on the opposite sidewall were treated with incrementally higher pressure water jets, prior to

un-reinforced shotcrete application. Adhesion test cores were drilled after the full 28-day shotcrete strength of 28 MPa had been reached and these were then pull tested according to the Swedish standard test method SS137243 (1987), see Figure 2. This is an *in situ* test that requires the drilling of a core though the shotcrete and an equivalent depth of rock. A friction grip ring is then placed around the core stub and coupled

		Adhes	Adhesion strength (MPa)			Failure surface		
Scaling/ cleaning method	No. of tests	Mean	Median	Std. dev.	At bond	In Rock		
Normal	41	0.21	0.08	0.27	20%	80%		
Water jet	24	0.61	0.68	0.45	42%	58%		

Table 1. Shotcrete adhesion test results at Kiruna.

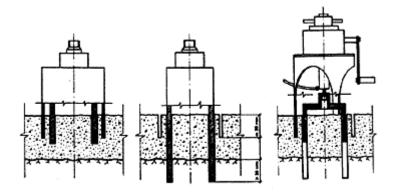


Figure 2. Adhesion testing arrangement.

to a tension device and the grip of the ring becomes stronger as the applied tension increases.

The high pressure water jet cleaning of the rock test panels showed a transition in effectiveness at about 3000 psi (or approximately 20 MPa) with fist sized and larger loose rocks starting to be removed. At 6000 psi the operator was bombarded by rock particles in a sandblast type effect, whilst the outer layer of the concrete test panel (28-day concrete strength of 26 MPa) was removed leaving etch lines on the surface.

Adhesion test results from the rock panels were inconclusive as failure occurred within the rock in all cases and along parting planes in the gneiss fabric. Thus the bond strength of the shotcrete rock interface was not effectively tested on the rock panels, although it was greater than the tensile strength of the wallrock. However, the results from the concrete panel tests (Figure 3) shows a fourfold increase in bond strength at 20 MPa water pressure, compared to normal watering down pressures of 0.7 MPa, with no significant increase at higher pressures.

Further studies by Kuchta et al (2003) compared the volume and size gradation of material scaled by manual and hydro scaling techniques in the same rock type at the

Army Tunnel; and reported a UDEC numerical simulation study of the rock-shotcrete interface characteristics on the support capacity of shotcrete. Hydro scaling removed generally finer material, 50% less than 38 mm (1.5 inches) as opposed to 10% by hand scaling; whilst the modelling showed that the tensile bond strength between the shotcrete and the rock surface was the key parameter for optimising the required thickness of the shotcrete layer. It was shown that poor interface strength could be overcome by increased shotcrete thickness, although this is a poor economic solution compared to proper cleaning of the surface. Together, these studies reinforced the importance of rock surface preparation, the cleaning effect of hydro scaling and the usefulness of adhesion testing.

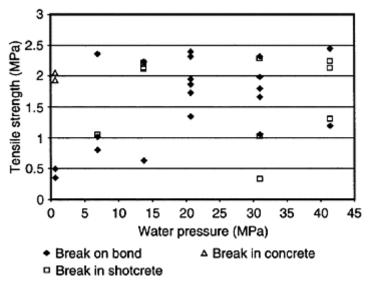


Figure 3. CSM adhesion results for concrete panels.

2.3 Other hydro scaling studies

The Swedish construction company Skanska and the Falconbridge Sudbury Operations have also undertaken trials with hydro scaling that were inconclusive though still provide useful insight.

In late-1988 at the Halandsâs tunnel project, Skanska mounted a pump and water jet on a shotcrete spraying machine in an attempt to establish whether hydro scaling was economically competitive with the mechanical scaling methods normally used in such situations. The pump was rated to spray water at a pressure of 14,000 psi (98 MPa) but the boom could not handle this force. A 60 MPa water jet was found to be very effective although a 30 MPa pressure was recommended for future use. Adhesion tests did indicate an increase in bond strength but this was not very conclusive and possibly due to the variety of rock types encountered in the tunnel and their varying state of weathering. Reduced scaling times and reduced damage to rock and equipment were evident from the Skanska trials. However, under-break occurred with some large blocks having to be removed by mechanical scaling. It was concluded that high pressure water could not completely replace mechanical scaling but that it was a good complement to this.

Falconbridge assessed hydro scaling as surface preparation for the Tekflex water based membrane liner using a 25 MPa pressure rated pump mounted to the spraying boom for the liner. Small scale loose up to 500 kg was easily removed in this trial and, as such, it was concluded that mechanical scaling was not mandatory for use with liners. Conversely, water-jet scaling was not recommended for use with mesh in very blocky conditions.

3 WA GROUND CONTROL PRACTICE

In Western Australia the installation of mesh or equivalent screen material is required, by the Code of Practice for Surface Rock Support, to prevent rock falls from between rockbolts in all excavations greater than 3.5 m high, MOSHAB (1999). That is, unless a geotechnical risk assessment by a suitably qualified person indicates that screen is not necessary to maintain safe working conditions; and such a judgment would require periodic re-assessment and check scaling to remain current. Surface support is required for all walls greater than 3.5 m above the floor, as manual scaling is considered problematic above this.

It is general practice in WA mines for mesh to be installed with bolts up to the face in development headings. This task is usually done by the development jumbo using friction bolts, after scaling or 'rattling' the face with the boring bit on the end of the boom. The rattling process is effective but in most ground conditions can lead to overbreak and unnecessary impact damage to the surrounding wall rock. Scaling is not a design function of the jumbo and causes increased wear and tear and increased maintenance costs for the jumbo. Due to the ease of split set type friction bolt installation with a jumbo there is a tendency for this type of bolt to be relied upon for long term ground support, a purpose for which it is not inherently suited.

To increase the life-span and capability of jumbo mesh and bolts, the grouting of friction bolts is undertaken by some mines and the current practice at Waroonga is to over-drill the split sets for later grouting with cable inserts. Other strategies used include jumbo meshing with resin bolt installation, for which there are several types of thicker walled tubular bolts available, or the subsequent installation of additional cement grouted rebar. Except for resin bolting, all of these strategies require the scheduling of additional ground support activities. Depending on the environment and required life span, corrosion may necessitate some degree of rehabilitation in all cases. In this regard, it is significant to note that the cost of decline rehabilitation at Agnew's Redeemer mine, which closed in early-2001, prohibited the extension of mining to exploit the deeper identified resource there.

4 WAROONGA TRIALS

Trials of ICS had been considered before at Agnew, but were only implemented when Byrnecut Mining, the underground contractor, in conjunction with Jetcrete Australia, proposed the combined use of hydro scaling.

4.1 Initial single heading trial

The objective of the initial trial was to demonstrate the feasibility of using hydro scaling and ICS to replace conventional jumbo rattling and mesh installation with friction bolts, as per the normal and trial mining cycles outlined in Table 2. The expectations of the trial were that it would result in improved long term ground control, increased jumbo availability/ productivity and development rates; as well as reduced jumbo operating costs, less check scaling, rework and rehabilitation of ground support.

4.1.1 Main Lode Decline ground conditions and support

The initial trial was conducted in the Main Lode exploration decline, located in the hangingwall rock mass that consists of well bedded, metamorphosed sandstones and siltstones. The bedding dips at 65° and strikes sub-parallel to the direction of decline advance. As such, the orientation of the decline is adverse and the time dependant loosening of bedding slabs had been experienced on both sidewalls, either by sliding (see Figure 4) or toppling, in the early stages of decline development.

Fume clearance period after firing of heading Re-entry and watering down for dust suppression						
Normal cycle	Trial cycle					
Bog the heading	Bog heading to grade line					
Jumbo set up and delivery of mesh & bolts	Hydro scale with modified Spraymec					
Rattle backs and walls	Agitator truck to decline & spray fibrecrete					
Bolt and mesh	Rattle down the face					
Rattle the face	Start boring the face top down					
Scratch-back	Re-bog to the floor & scratch-back					
Mark up, bore face bottom-up	Finish boring					
Hand scale, charge and fire the heading	Charge and fire the heading					

Table 2. Change of development cycle for the initial trial.

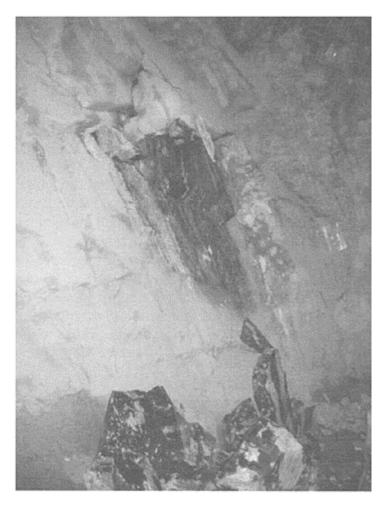


Figure 4. Time dependant deterioration and failure of bedding slabs in the Main Lode Decline, February 2003.

The standard ground support profile for the 6 m×6 m arch decline (WAR SP1, see Fig. 5) allows for mesh to a height of 3.5 m, with spot bolts as required below the mesh. This had been upgraded, before the trial took place, with an additional sheet of mesh on each sidewall to contain the unstable slabs.

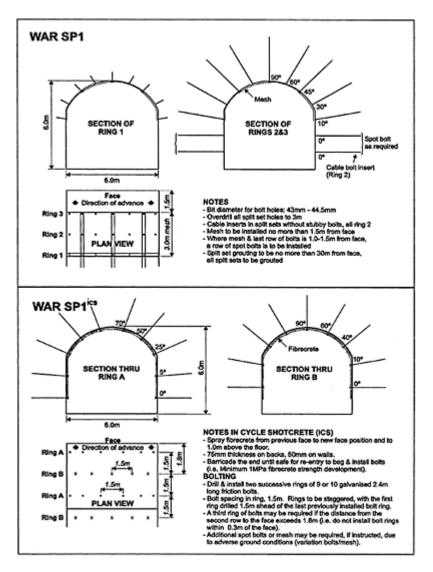


Figure 5. Waroonga Main Lode decline Ground support profiles: WAR SP1 for conventional scaling, mesh and bolts; and WAR SP1^{ICS} for extended mine-wide ICS trial.

A nominal 75 mm thickness of shotcrete reinforced with polypropylene fibres was specified for the trial. The fibrecrete was to be sprayed on the backs and sidewalls to the same height as the conventional mesh coverage. For the purposes of the initial trial, there

was no attempt to optimise the bolting pattern, which remained as per the WAR SP1 profile, allowing mesh to be installed conventionally should it be required for any reason.

The fibrecrete used for the trial was a standard 32 MPa Jetcrete mix design, with quantities per cubic metre as listed below:

GP Cement	420 kg
Aggregate=7 mm	550 kg
Coarse sand	360 kg
Fine sand	770 kg
Bar chip synthetic fibres	7 kg
Water	175 litres
Accelerator (Sigunit-L50AF)	16–25 litres

4.1.2 Equipment and method

A water pump and additional nozzle were attached to a Normet Spraymec 6050 shotcrete machine to modify this for hydro scaling, prior to fibrecrete placement. The pump is capable of delivering 501/min at a pressure of 6000 psi or 41 MPa. However, based on the findings of the previous hydro scaling studies the pump and nozzle were pre-set to deliver at a maximum pressure of 3000 psi or 20 MPa. Figure 6 shows hydro scaling in progress with the operator controlling the boom and nozzle from the control unit, 'worn' at waist height and mounted from a shoulder harness, as for shotcrete spraying.

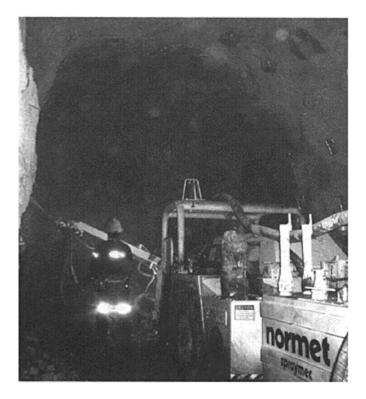


Figure 6. Hydro scaling in progress.

The wet mix fibrecrete product was supplied from a batch plant in Leinster, 30 km away, with delivery by agitator truck direct to the Spraymec hopper at the work location. After the completion of fibrecrete spraying, a nominal strength gain of 1 MPa was set as the reentry requirement before bolting commenced. This was tested with a penetrometer was used to confirm the 1 MPa strength that generally coincided with the stage at which visible signs of fibrecrete curing, 'white tips', could be seen.

The thickness of the shotcrete layer was tested during spraying by probing with a depth gauge attached to the end of the spray nozzle. Quality control of the fibrecrete mix was verified by compressive strength testing of cylinders prepared from the fibrecrete product on batching. Also test panels are sprayed at the point of application, from which core samples are taken for similar compressive tests. In addition, Jetcrete sourced drilling and pull test equipment for adhesion testing to be done.

4.2 Initial trial evaluation

4.2.1 Hydro scaling

Loose material removed by hydro scaling was generally of scat size, with a maximum dimension of up to 300 mm. However, the mobility of the robotic arm of the Spraymec and its skilled operation is important as the water jet needs to be directed into the cracks and joints to create the pore pressure required to propagate cracks and dislodge blocks. It was observed that some larger slabs, up to 1 m long and weighing up to 800 kg were also dislodged.

It is inferred that the water jetting tended to remove only the looser, blast fractured material from the sidewalls. In comparison, mechanical jumbo scaling dislodged blocks up to $1.5 \text{ m} \times 1.5 \text{ m} \times 0.5 \text{ m}$ or approximately 4.5 Tonnes. There were no large or overhanging slabs that could not be removed by hydro scaling and it was not necessary for the jumbo to further scale the backs and walls of the decline in the trial section.

4.2.2 Cycle times

Time and motion studies were conducted during the eleven day trial, during which time sixteen cuts were taken at the decline face to achieving 48.3 m advance. Similar studies were conducted in a conventional mesh and bolted development end for comparison purposes. Apart from ground support activities, all other activities are assumed to take the same amount of time to perform. The summarised results of this study are:

- Conventional bolts & mesh (six sheets)

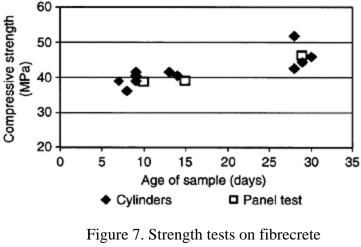
Time to jumbo scale	33 minutes
Time to bolt & mesh	158 minutes
Total support time	191 minutes
Hydro scale & in-cycle fibrecrete	
Time to hydro scale	21 minutes
Time to fibrecrete	37 minutes
Time to bolt (ave. 23 bolts)	59 minutes
Total support time	117 minutes

The average cycle time per cut for ground support was reduced by 38% for the hydro scaling and ICS, a 74 minute saving over the average time for the comparable six sheet mesh profile installation. It was estimated that jumbo availability would increase by five hours per day, equating to a potential 25% increase in jumbo productivity.

4.2.3 Fibrecrete

The fibrecrete strength specification of 32 MPa at 28 days was well exceeded by all the core samples taken from test panels, as well as by the single panel test, see Figure 7. The

actual thickness of fibrecrete was variable, generally greater than the 75 mm specified and up to 130 mm thick. This was due to the minimum product delivery constraint of 5 m^3 that resulted in the full delivered amount being sprayed in each cut.



cylinders.

The shotcrete adhesion tests were difficult to perform. The single bit drill used for coring *in situ* was mounted on the basket of an elevated work platform and proved unable to successfully core to the required depth of rock after penetrating the usually thick layer of fibrecrete. Also, by the time the drilling equipment was available and test sites successfully cored, the fibrecrete was 150 days old. The limited results given in Table 3 may not therefore be a valid direct comparison with previous studies, but they do indicate the achievement of significantly higher bond strengths at Waroonga than those previously reported.

4.2.4 Profile control

One of the more dramatic outcomes of the trial was the improvement in the arch profile of the Main Lode decline and reduction in overbreak, as Figures 8 and 9 demonstrate. The average overbreak before the trial

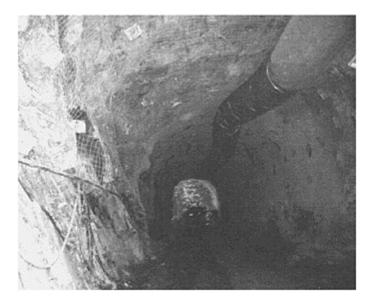


Figure 8. View looking down the Main Lode decline from before the start of the trial section.

Scaling	Shotcrete thickness	Locat	ion of t	failure		Tensile strength
method	(mm)	Bond	Rock	Pull cap	Shotcrete	(MPa)
Hydro	10		Х		X	5.8
Hydro	70	Х				3.98
Hydro	75	Х				3.18
Hydro	30	Х				2.7
Hydro	50				Х	9.6
Hydro	75			Х		11.3
Hydro	70	Х				2.83
		Avera	ge Hydr	o Scaled	Strength	5.63
Jumbo	80	Х				4.27
Jumbo	35	Х				4.77
Jumbo	45	Х				4.37
Jumbo	120			Х		6.3

Average Jumbo Scaled Strength

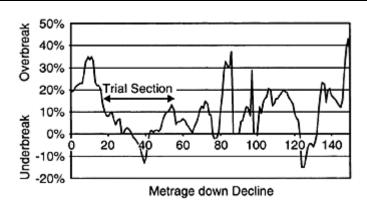


Figure 9. Design profile compliance, Main Lode decline during the initial trial.

section was 26%; this was reduced to 6% during the trial and subsequently increased again to 13% after the trial.

4.2.5 Initial trial conclusions

The hydro scaling was considered to be very effective for the ground conditions encountered and combined with in-cycle fibrecrete placement was capable of achieving all the perceived benefits identified, namely:

- increased jumbo productivity and reduced damage to both equipment and wall rock;
- improved tunnel profile with overbreak minimised;
- improved long term ground support, quality and duration;
- an improved underground environment with reduced ventilation airflow resistance;
- reduced check scaling requirements;
- reduced personnel exposure;
- reduced future ground support re-work and rehabilitation.

4.2.6 Financial evaluation

During the initial trial the delivered cost of fibrecrete sourced from Leinster was \$583 per m³ and inclusive of equipment and labour, the cost was \$919 per m³. This is obviously far higher than the cost of meshing and bolting. However, with the vision of fully integrating the method into all development activities on the mine, there are many other factors to be considered in evaluating the costs and benefits.

Byrnecut submitted a three year underground contract tender and factored hydro scaling and ICS into this with the following assumptions:

- the method would be applied in all waste development at Waroonga and a portion of the ore drive development where conditions warranted fibrecrete;
- an additional six man crew plus supervisor would be integrated into the work force and used for other duties when available; and
- a dedicated batch plant would be established and use locally produced aggregate.

For the hydro scaled and ICS development component of the tender alone there was a cost increase of 6–8%, depending on the size of the excavation. However, for the overall contract this equated to a 4.8% cost increase.

5 FURTHER EVALUATION AND TRIALS

5.1 Contract innovation committee

On completion of the financial evaluation of the initial trial the Contract Innovation Committee, with representatives from Agnew, Byrnecut and Jetcrete reviewed all aspects of the supply, application and integration of the method on a mine-wide basis. This indicated that compared to the base case with no hydro scaling and ICS, a near-cost neutral or better financial result appeared feasible; and it was resolved to undertake a further extended mine-wide trial.

Before undertaking a three month mine-wide trial that commenced on 1st November 2003, the use of hydro scaling in ore drives and the suitability of locally produced aggregate were both evaluated. It was established that there was no benefit from using hydro scaling to replace jumbo scaling where mesh and bolts are installed, as significant additional scaling was necessary when the jumbo re-entered the heading; an observation that agrees with the findings of Kuchta (2003). However, the application of ICS in the ore drives, which are mined under geological control, was considered possible if digital photographic techniques for geological mapping were successfully introduced.

The crushing of run of mine waste from Waroonga at the Agnew mill and it's evaluation as a part of a revised mix design indicated that the material cost of fibrecrete in the initial trial, \$583 per m³, would be reduced to \$313 per m³.

5.2 Ground support design

A review of the available literature was undertaken prior to finalising the recommended ground support profiles for mine-wide trials. Most authors recognise that the support and reinforcement effects of shotcrete are difficult to quantify and that the available design guidelines are empirical or "rule of thumb". The support profiles recommended have been compared to the available design guidelines, as reviewed by Langille (2001); and with the deterministic methods proposed by Barrett & McCreath (1995), who comment that "shotcrete's full potential as a method of ground support is rarely exploited and frequently it is not properly integrated into the excavation-support cycle".

5.2.1 Boltless flbrecrete

Inco has implemented boltless fibrecrete ground support regimes, with the initial trial application being at the Stobie mine, Espley *et al* (2001). At Stobie 75 mm fibrecrete thickness on the backs was tapered to 25 mm on the walls, but was not entirely successful for long term support purposes as the sidewalls sustained equipment damage and significant deterioration. However, increasing the fibrecrete thickness on the walls, with a 63 mm application on both walls and back was successful and eliminated the sidewall deterioration. Initial re-entry times of 12 hours were reduced to 8 hours through the use of accelerators to increase the early strength of the fibrecrete.

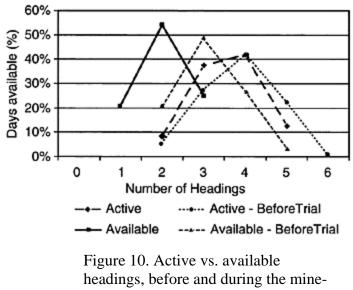
5.2.2 Support profile for mine-wide trial

The initial trial at Waroonga had shown that the integration of hydro scaling and shotcrete into the development cycle was achievable and it was proposed that the maximum benefit from the use of this method would be gained if the full potential of shotcrete as ground support were realised.

The adhesion tests completed to date demonstrate the beneficial effect of hydro scaling prior to fibrecrete application and indicate adhesive bond strengths that exceed the generally accepted industry standard of 1–2 MPa. The achievement of bond strengths exceeding 1.5 MPa is a key support design assumption and ensures that the shear strength of the shotcrete layer rather than it's flexural or tensile strength will be fully mobilised as rock mass reinforcement, Barrett & McCreath (1995). The shear strength gain of the fibrecrete, to determine safe re-entry times, and the achievement of design cover thickness are then the key quality control issues if the adoption of a boltless ground support profile were to be considered.

Historically, shotcrete thickness has been arbitrarily specified and rarely well controlled in application, due to the irregular drill and blast excavation process. Improved methods of specifying and evaluating shotcrete thickness have been advocated; these are the "cover techniques" of Windsor (1998). It is suggested that full utilisation of the support capability of fibrecrete in the manner envisaged will require the adoption of these techniques for specification and measurement for quality control purposes. Likewise, the development of a test procedure to verify the *in situ* shear strength of fibrecrete is also considered necessary to meet this goal.

In view of these issues, it was not considered prudent to attempt a boltless fibrecrete regime for the extended trial. Nevertheless, the support strategy recommended utilised the fibrecrete layer as the principal component of the permanent support system. A 75 mm fibrecrete thickness was specified on the backs with 50 mm on the walls down to 1.0 m above the floor in the declines and other long term infrastructure excavations. In short term excavations the fibrecrete on the walls generally extended down to 3.0 m above the floor. Ungrouted, galvanised friction bolts were



wide trial.

installed after the nominal 1 MPa fibrecrete strength had been achieved, to provide immediate support and allow re-entry. The spacing of the bolts, not now constrained by mesh size, was increased to a 1.5 m staggered pattern as shown in Figure 4.

5.3 Extended mine-wide trial

Despite some initial teething problems with the on-site batching of fibrecrete, the technical side of the trial was a success. However, the potential productivity benefits were never realised due to severe constraints on heading availability. There were several contributing factors causing the lack of available headings for ICS, most prominent was the lack of Main Lode heading availability that was still only a single heading for the duration of the trial. Since the initial trial progress in the Main Lode Decline had been slowed down by the intersection and ingress of significant quantities of water; this before the planned pumping capacity infrastructure had been established. During the course of the extended trial there were rarely four headings available for ICS during any 24-hour period, as Figure 10 shows; whereas four or five headings consistently being available would have utilised the equipment fleet and personnel to maximum effect.

The establishment of a leased batch plant and the introduction and training of personnel for the shotcrete crews was accomplished without problems. But frequent interruptions to fibrecrete spraying were experienced in the first few weeks of the trial due to nozzle blockages. These were due to oversize material being picked up when aggregate crushed to the required specification was transported from the mill to the batch plant. As a result, average the times for hydro scaling and ICS spraying in the first few weeks of the trial were 80–90 minutes. Towards the end of the trial period the average cycle time for ICS had been reduced to 52 minutes.

The presence of ammonia gas in the headings after ICS spraying and on re-entry did cause some concerns and minor delays. The ammonia was produced by reaction of the shotcrete with Anfo and although not toxic this did cause discomfort to some personnel; and those affected were advised to withdraw. Improved ventilation and care in preventing Anfo from being left in the heading after charge-up largely eliminated this issue.

Overbreak in the waste ends mined during the trial averaged 3% with variation between 1-4% only for each set of headings as measured per monthly broken tonnage. In comparison, for the six months prior to the trial the average overbreak was 6% and varied over a much larger range, from -8% to 13%.

5.4 Mine-wide trial results

The three month extended trial could not be considered a success as the main purpose, that of realizing the potential productivity benefits offered by hydro scaling and ICS, was not achieved. In fact, the Powerclass Jumbo used for waste development was underutilised as it had less work to do. It was disappointing that four headings were available on only a few days towards the end of the trial, with mostly only two available. The optimum availability to give the trial a fair chance would have been four or five headings being consistently available. Largely as a result of this, the average long term additional cost of the trial (compared to the rate for conventional mesh and bolts) was around \$400 per m.

The positive outcomes from the trial were that:

- Incorporation of the method into the mining cycle on a mine-wide scale was achieved.
- The alternative ground support regime introduced for the trial was successful.
- Heading profiles were improved.
- Hydro scaling and ICS cycle times were reduced against the initial trial base case.
- Fibrecrete product performance met or exceeded the required specification.
- All operators readily accepted the changes brought about by the trial.
- Development of intersections was faster and tidier than with conventional techniques.

6 CONCLUSIONS

The trials of hydro scaling and ICS at Waroonga have successfully demonstrated the benefits of this technique, which is a potentially best practice development for the mining industry. Lack of heading availability prevented a successful economic outcome to the extended mine-wide trial. However, it is still possible that the method will be adopted on a minewide basis at Waroonga when there is sufficient heading availability and when future ore drives are driven in ultramafic rock, for which it is anticipated that ICS will be required.

Hydro scaling is considered to be highly advantageous for any long-term shotcrete or fibrecrete application under most ground conditions. The development of a boltless fibrecrete ground support regime is thought to be possible with good surface preparation (i.e. hydro scaling); although the shear strength gain and achievement of adequate fibrecrete cover thickness are quality controls required for this, in particular for minimised re-entry times.

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Strength and stiffness of shotcrete-rock interface—a laboratory study

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ABSTRACT: Over the past several years Kiirunavaara underground iron ore mine in Sweden (LKAB) has been pursuing a study on the interaction between its surface rock support systems and the rock mass. One of the major areas of this study is the interaction between shotcrete and rock. The strength and stiffness of the interface significantly influence the effectiveness of shotcrete to support a rock mass. This paper presents the results of a series of laboratory tests namely direct shear, tension and compression, performed on shotcrete-rock interfaces to determine the strength parameters. The direct shear test, which formed the core of the experimental work, was conducted under low normal stresses in order to simulate field conditions as close as possible. The results show that the shear strength is in principal determined by the bond strength or the cohesion at normal stresses less than 1.0 MPa. Beyond 1.0 MPa the peak shear strength was markedly dominated by friction resulting in cohesion being less significant. After test assessment of the shear surfaces revealed that the steel fibres in the shotcrete appeared to contribute considerably to the frictional component. An interesting observation was the complex interaction at the interface and the mechanisms that controlled the peak strength which depended very much on the surface roughness, existence of natural flaws and the normal load.

1 INTRODUCTION

Over the past several years LKAB's Kiirunavaara underground iron ore mine in Sweden has been pursuing a study on the interaction of its surface rock support system and the rock mass. One of the major areas of this study is the interaction between shotcrete and rock. The actual interaction between the shotcrete and rock is quite complex as shown by for example, Holmgren (1979, 1985), Stille (1992), Stacey (2001) and Malmgren (2001). Although complex the interaction itself is after all dependent on the strength of the

interface and its governing mechanical properties, particularly the stiffness and friction coefficients. At a genuinely cemented interface the adhesion strength is important. External factors such as rock surface preparation and geometry of the rock surface on which shotcrete is applied have been found to affect the adhesion strength quite significantly. Malmgren (2001) has shown that the adhesion strength of the shotcrete-rock interface at Kiirunavaara underground iron ore mine was significantly increased when the rock surface was prepared by water-jet scaling.

Some of the early studies on the strength of shotcrete-rock interfaces were by Fernandez-Delgado et al. (1976) and Holmgren (1979). Since then a large and varied number of tests have been performed, including field studies and observations. However, due to the complexities of the shotcrete-rock interaction the various methods could only provide specific data for simple ground conditions. The direct shear test is one way of studying the strength of the interface and its mechanical properties. Thus this study mainly focuses on the direct shear test, with tension and compression tests being complementary. The direct shear test was conducted under low normal loads to simulate field conditions as close as possible. For most practical cases where shotcrete is used with rock bolts the normal load on shotcrete lining rarely exceeds 200–500 kN/m² (or 0.2 to 0.5 MPa).

Although no citations were made on past experimental work on the shear strength of cemented shotcrete-rock joints by direct shear test method a number of tests have been conducted on non-cemented concrete-rock joints. For example, by Johnston & Lam (1984), Lam & Johnston (1989), Kodikara & Johnston (1994), Seidel & Haberfield (2002), and Changwoo et al. (2002). Cater & Ooi (1988) studied shear hardening and softening behavior of genuinely cemented concrete-rock joints.

2 TEST SAMPLES

The initial preparation of the test samples were carried out at the Kiirunavaara underground iron ore mine facilities, from where the samples were collected. One advantage for preparing the samples at the mine site was that the shotcreting technique used in the actual operation was used to prepare the test samples.

The rock samples mainly comprised magnetite iron ore and trachyte waste rock. Magnetite is the principal iron ore mined at Kiirunavaara and trachyte is the waste rock at the footwall side of the ore-body. The rock pieces were fresh and taken directly from collapsed material from the roof and the walls. The average uniaxial compressive strength of magnetite is 130 MPa and for trachyte, 200 MPa.

The surface roughness estimated in x and y directions, using Barton and Choubey's (1977) Joint Roughness Coefficient (*JRC*) Chart, ranged from 1 to 13 for the rock specimens collected. All magnetite surfaces registered *JRC* values from 1 to 3 along with 50% of the trachyte samples. The other 50% of the tracyte samples registered *JRC* values from 9 to 13.

Before shotcreting, the rock pieces were cleaned with water to achieve good adhesion and then placed inside wooden troughs and shotcreted. The shotcrete mixture used is shown in Table 1. After curing for 28 days the test samples, shown in Figure 1, were extracted by coring. Those for direct shear test were cored using 180 mm inner diameter diamond drill bit, while those for tensile and compression test were obtained using 94 mm inner diameter diamond drill bit. The diametrical specifications for the specimens were predetermined to comply with the laboratory test equipment and test method standards.

The final preparations of the test samples were done in the laboratory at Lulea University of Technology. These preparations included edge preparation of the tensile and compression test specimens, and molding of the direct shear specimens in concrete. A typical final direct shear test sample, ready for testing, is shown in Figure 2. The actual test specimen is encapsulated by a pre-mixed rapid hardening concrete, capable of achieving its full strength within 7 days. These samples were prepared inside stiff metal molds to conserve stiffness, shape, dimension

Table 1. Shotcrete ingredients, wet-mix method.

Ingredient	Ratio
Cement (kg/m ³)	506
Silica (kg/m ³)	20
Aggregate, dry weight (kg/m ³)	1435
Steel fibre, Dramix 65/35 (kg/m ³)	50
Slump (mm)	150
Water cement ratio (%)	38

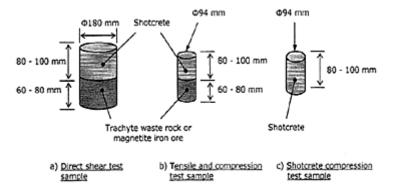


Figure 1. Test samples.

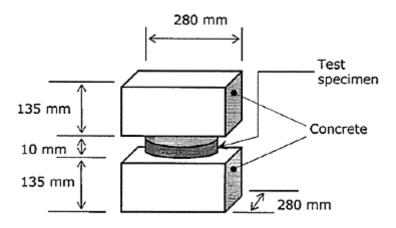


Figure 2. Direct shear test sample.

and clean finish. Circular clamps were used to center the specimens during molding so that the joints coincide with the shear plane. The final dimensions of test blocks were 280 mm×280 mm×280 mm with 10 mm clearance around the joint to allow freedom of shear and lateral displacement. The average age of shotcrete at the time of testing was 50 days.

3 EXPERIMENTATION

3.1 Direct shear

A stiff servo controlled direct shear machine, shown in Figure 3, with a loading capacity of 500 kN for both normal and shear forces was used to perform the direct shear tests. For control and data acquisition the machine was equipped with two computers, one for normal control and the other for data acquisition.

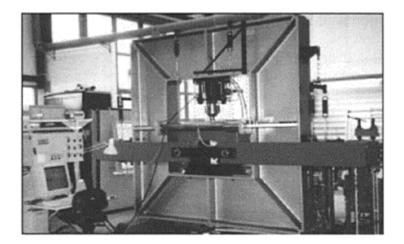


Figure 3. Luleå University of Technology's stiff servo controlled direct shear machine.

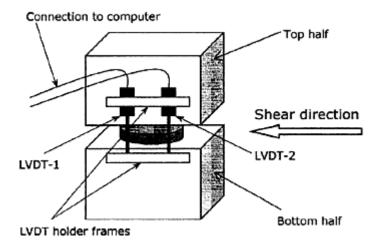


Figure 4. Experimental setup. LVDTs are systematically placed on the front and back of sample. LVDTs 3 and 4 are at the back of the sample.

Prior to the actual tests, trial tests were performed on 4 samples for sensitivity evaluation and identification of suitable test conditions. This also included testing for eccentricity, tilt and rotation of the test sample inside the shear box. On the basis of these tests the normal load range was set at 1 to 40 kN (0.04 to 1.57 MPa), which was sufficient to avoid experimental uncertainties and at the same time simulate field conditions as close as possible. The shear displacement rate was set at 0.1 mm/min. The test was carried out under constant normal load conditions.

To avoid disturbance to joints a crane was used to mount the samples inside the shear box. After mounting, the LVDTs for monitoring dilation were prepared and systematically glued around the samples as shown in Figure 4. When ready for testing the normal load was raised steadily to the required level and allowed to stabilize before applying the shear force. The normal force was held constant while the shear force was applied. Results recorded include, cumulative shear force (in kN), shear displacements (in mm) and normal displacements (in mm).

A total of 38 samples were tested after sorting into 3 groups according to the *JRC* values and rock type.

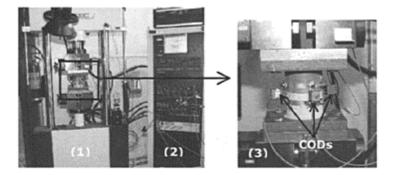


Figure 5. Experimental setup for joint tensile test. (1) Dartec hydraulic testing machine, (2) Dartec electronic system and (3) enlarged view of sample sandwiched between loading cells.

Group 1 consisted of 20 shotcrete-magnetite samples with *JRC* values of 1 to 3, group 2 comprised 9 shotcrete-trachyte samples with *JRC* values of 1 to 3, and group 3 consisted of 9 shotcrete-trachyte samples with *JRC* values of 9 to 13.

3.2 Joint tensile test

The purpose of tensile tests on shotcrete-rock joints was to determine the adhesion strength of the joints. A total of 7 samples (3 shotcrete-tracyte and 4 shotcrete-magnetite) were subjected to a tensile test using a Dartec low capacity (50 kN) servo controlled hydraulic testing machine equipped with Dartec electronic system. The experimental set up is as shown in Figure 5. The joint displacements were measured using four Crack Opening Displacement (COD) gauges evenly placed around the interface. To avoid uncontrolled failure the test was conducted with a displacement rate of 0.0001 mm/s.

3.3 Joint compression test

Compression tests on shotcrete-rock joints were performed to determine the joint compressive strength (*JCS*) and the joint normal stiffness (K_n). The test was conducted using an Instron servo controlled hydraulic testing machine, also equipped with Dartec electronic system for data acquisition. To measure joint closure 4 COD gauges were evenly placed around the interface akin to the setup in Figure 5. The test was controlled with a displacement rate of 0.005 mm/s. A total of 4 samples were tested.

3.4 Shotcrete compression test

Compression tests on shotcrete specimens were mainly to determine the uniaxial compressive strength of the shotcrete used in preparing the jointed test specimens. A total

of 12 shotcrete specimens were compressed using the same equipment used in joint compression tests, except that the deformations were monitored by an LVDT.

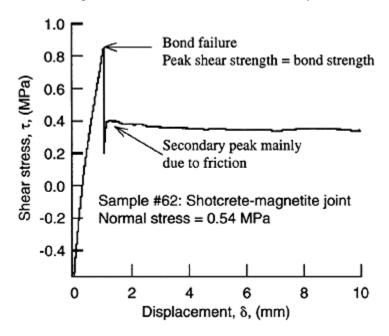


Figure 6a. Typical test result for joints with good adhesion. The peak shear strength is equal to the bond strength.

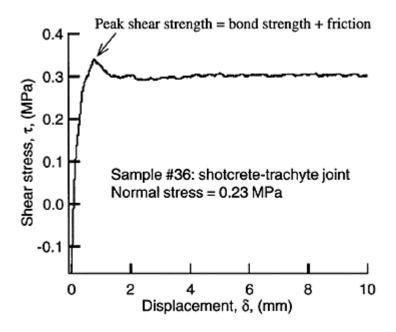


Figure 6b. Typical test result for a joint with either poor adhesion or joint tested at higher normal stress. The peak shear strength is dictated by both the bond strength and friction.

4 TEST RESULTS

4.1 Direct shear test results

Since the major part of this study was devoted to direct shear test emphasis will be primarily on direct shear test results.

4.1.1 Determination of peak shear strength

Because of the nature of the test results it is necessary to explain how the peak values for shear and residual strengths were obtained from the stress-displacement plots. Typically two types of observations were made as shown in Figures 6a and 6b. Figure 6a represents the result of a joint with good adhesion. The shear stress was observed to increase steeply until the bond failed and at that point the shear stress dropped sharply to a level where the displacement could be controlled. Then the shear stress increased again until a new peak is attained but is less than the bond failure stress. Clearly, the peak shear strength corresponds to stress at which the bond failed, which is the bond strength. The second peak attained was mainly due to friction. The residual strength is the residual value attained in the shearing of the unbonded joint. The two stage phenomenon of Figure 6a, is the result of using low shear displacement rate and low normal loads.

Figure 6b represents the result of joints with either poor adhesion or joints being tested at higher normal stresses. Here, the shear stress kept increasing until the peak shear strength is attained and then the stress drops to a residual value. In this case the peak shear strength is a function of both the bond strength and the friction. There were no clear indications of bond failure on the stress-displacement plots for such cases. Conversely they were noted and recorded during the testing process by visual and audible observations. Luckily, most of the bonds snapped with audible bangs, but whether these bangs indicate full or just partial fracture were difficult to verify. Interfaces that lost their bonds during initial application of the normal loads were treated as having zero bond strength.

4.1.2 Test results for shotcrete-rock joints with JRC=1-3

Results for shotcrete-trachyte and shotcrete-magnetite joints with *JRC* value of 1 to 3 were combined since they were similar. Table 2 shows the test results for these joints. The peak and residual strengths were determined using the procedures described in the preceding section. The peaks corresponding to stress at which the bonds failed (i.e. bond strength) are marked with a \dagger (dagger) symbol. Note that the term bond strength is used herein to describe the strength of the bonding between shotcrete and rock resulting from the direct shear while adhesion strength is used for those resulting from tensile test. This is mainly because of the different bond failure mechanisms involved in the two tests, which will be discussed later. The shear stiffness (K_s) values were determined from the tangent at 50% of the peak stress on the stress-displacement plot.

Figure 7a shows the peak shear strength plot for the interfaces. A distinction is made between the peaks corresponding to the bond strength and those resulting from combined effect of bond strength and friction. A linear fit could not be done because the peaks attained are clearly the result of different mechanisms. As can be seen, at normal stresses less than

Sample #	Normal stress (MPa)	Joint type	Peak shear strength (MPa)	Residual stress (MPa)	Shear stiffness (MPa/m)
58	0.03	S-M*	0.18	0.00	0.29
5	0.03	S-T**	0.24†	0.00	0.14
65	0.11	S-M	0.18	0.00	0.33
39	0.13	S-T	0.11	0.00	1.50
51	0.23	S-M	0.42†	0.00	2.25
36	0.23	S-T	0.34	0.29	0.65
33	0.27	S-M	0.28†	0.20	1.30

Table 2. Test results for shotcrete-rock joints, *JRC*=1–3.

52	0.29	S-M	0.69†	0.39	0.64
32	0.31	S-T	0.35†	0.21	0.98
53	0.33	S-M	0.35†	0.31	0.67
31	0.35	S-T	0.42†	0.28	0.50
57	0.42	S-M	0.35	0.33	0.63
6	0.42	S-T	0.71†	0.35	0.24
69	0.48	S-M	0.61†	0.49	0.93
62	0.54	S-M	0.85†	0.34	2.60
66	0.62	S-M	0.63	0.50	1.16
37	0.62	S-T	0.90	0.49	1.60
61	0.82	S-M	0.57†	0.63	2.33
38	0.82	S-T	1.15	0.66	0.75
59	0.90	S-M	0.79	0.63	0.80
64	1.21	S-M	1.21	1.02	2.10
34	1.21	S-T	1.14	0.92	2.00
56	1.41	S-M	1.53	1.06	2.70
63	2.00	S-M	1.81	1.44	1.36
54	2.39	S-M	2.12	1.76	3.00
68	3.57	S-M	3.07	2.40	3.17

*S-M: Shotcrete-magnetite joint.

**S-T: Shotcrete-trachyte joint.

†peaks corresponding to bond strength.

1.0 MPa the shear strength is mainly determined by the bond strength and beyond 1.0 MPa it is determined by a combination of bond strength and friction. It is the shear strength at normal stresses less than 1.0 MPa that is of interest to this study because it has practical significance to shotcrete when it used as surface rock support. In most practical cases where shotcrete is used with rock bolts the usual or perhaps the maximum normal stresses seldom exceeds 0.2 to 0.5 MPa. Therefore the shear strengths at normal stresses up to 1.0 MPa are isolated and plotted in Figure 7b. Those shear strengths that resulted from the combined effect of bond strength and friction are omitted so that the significance of the bond strengths can be clearly shown. Also, the failure mechanism observed for those cases were quite complex. Thus, Figure 7b shows that at genuinely cemented shotcrete-rock interfaces the shear strength is effectively determined by the bond strength for the normal stresses anticipated in practical cases. There is a notable scatter of the bond strengths, which obviously reflects the quality of adhesion between the shotcrete

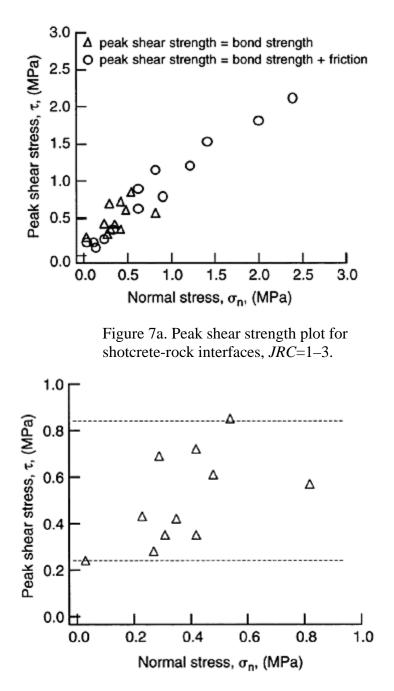


Figure 7b. Plot of peak shear strengths corresponding to bond strengths for

shotcrete-rock interfaces for normal stresses less than 1.0 MPa, *JRC*=1–3.

and the rock. A linear fit has been tried but that resulted in a correlation coefficient of 0.31, which is not very good. Hence the best that could be done was to average the bond strengths which resulted in an average of 0.51 MPa, with the upper and lower bounds approximately equal to 0.24 and 0.85 MPa respectively These upper and lower bounds are shown as dashed horizontal lines in Figure 7b. The average bond strength can be considered as the cohesive strength of the interfaces.

Figure 7c shows the plot of the residual strengths. The residual strength values were obtained as described in the previous section. From this figure the residual friction angle determined is 35.4° . The peak

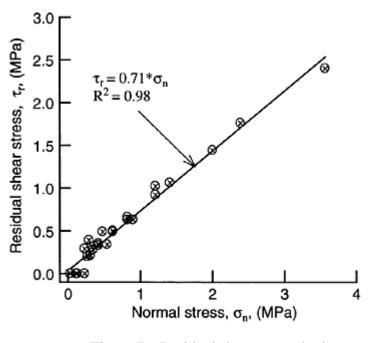
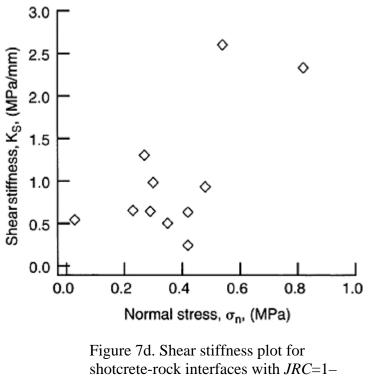


Figure 7c. Residual shear strength plot for shotcrete-rock interfaces, JRC=1-3.



3.

friction angle determined by using the secondary peaks, which occurred after the bonds had failed (see Figure 6a) is 40.0°.

Figure 7d shows the shear stiffness. As noted earlier the stiffness values were determined from the tangents at 50% of the peak shear strength, in this case the bond strength. Thus the shear stiffness values are essentially the stiffness of the bonding between the shotcrete and the rock. As seen there is almost no relationship between the stiffness and the normal stress. Therefore an average value of 1.0 MPa/mm was obtained for these interfaces for the given normal stress range.

Sample #	Normal stress (MPa)	Joint type	Peak shear strength (MPa)	Residual stress (MPa)	Shear stiffness (MPa/m)
12	0.13	S-T*	0.22	0.20	0.30
14	0.29	S-T	0.37	0.00	0.80
16	0.42	S-T	1.24†	0.49	0.80
17	0.50	S-T	1.85†	0.75	3.00

Table 3. Test results for shotcrete-rock joints, *JRC*=9–13.

15	0.54	S-T	1.12†	0.55	1.10
11	0.66	S-T	1.23†	0.94	0.95
13	0.82	S-T	1.42†	1.06	3.50
19	1.13	S-T	1.23	0.80	2.00

*S-T: Shotcrete-trachyte joint.

† Peaks corresponding to bond strength.

4.1.3 Test results for shotcrete-rock joints with JRC=9–13

The test results for shotcrete-rock joints with *JRC* of 9 to 13 (this group was mainly shotcrete-trachyte) are shown in Table 3. As for interfaces with *JRC* of 1 to 3 the shear strengths corresponding to the bond strengths are marked with a \dagger (dagger) symbol. The strength plots for this group of interfaces are shown in Figures 8a to 8c. As before a distinction is made between the peak shear strengths corresponding to bond strengths from those resulting from combination of bond strength and friction in Figure 8a. In Figure 8b the shear strength corresponding to the bond strength for normal stresses less than 1.0 MPa are shown. A similar observation as for interfaces with *JRC* of 1 to 3 was seen. Thus the bond strength was estimated with an average bounded by lower and upper limits. The average is 1.37 MPa and the upper and the lower bounds are 1.24 and 1.85 MPa respectively. Again this average bond strength can be considered as the cohesive strength of shotcrete-rock joints with *JRC* of 9 to 13.

Figure 8c shows the residual strength plot for these interfaces. Using this figure the residual friction angle determined is 39.0° . The peak friction angle determined in the same way as for interfaces with *JRC*=1–3 was found to be 47.2° . Figure 8d shows the shear stiffness plot for the interfaces whose shear strength corresponded to the bond strength. Again no trend is seen and therefore an average stiffness of 2.0 MPa/mm is obtained.

4.1.4 After-test shear surface assessment

After each test the shear surfaces were assessed for distinguished shearing patterns. The general observations can be summarized as follows:

1. For the shotcrete-magnetite interfaces shearing mainly occurred on the magnetite surface as

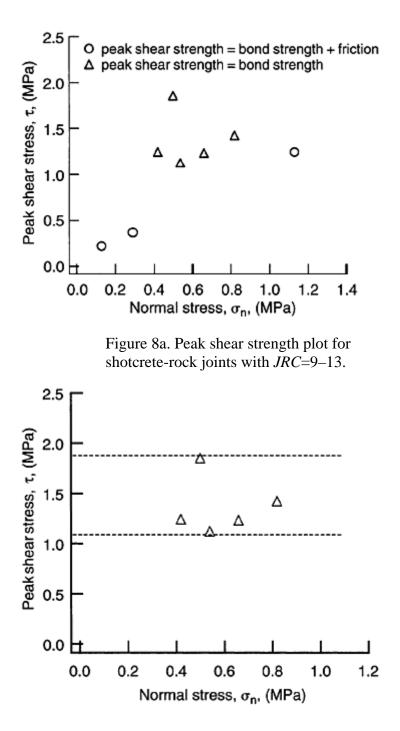


Figure 8b. Plot of peak shear strengths corresponding to the bond strengths for shotcrete-rock joints, JRC=9-13.

evidenced by frequent plugging, chipping and crushing of the magnetite surface.

- 2. For the shotcrete-trachyte joints shearing was mainly clean and frequently occurred along the interface without significant chipping of the rock surface as occurred with the magnetite.
- 3. The intensity of surface damage was more significant on magnetite surfaces than on trachyte. This may perhaps be attributed to the different hardness factors of the two rock types.
- 4. Asperity shearing and over-ridding were quite obvious for shotcrete-trachyte joints having *JRC* values of 9 to 13. Frequent polishing of shotcrete surfaces were observed in areas of full contact

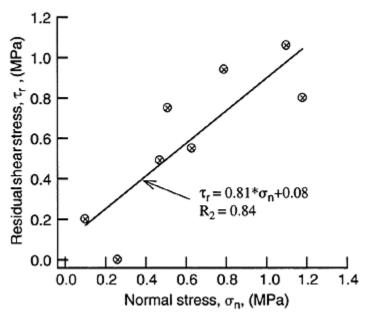


Figure 8c. Residual strength plot for shotcrete-rock joints with *JRC*=9–13.

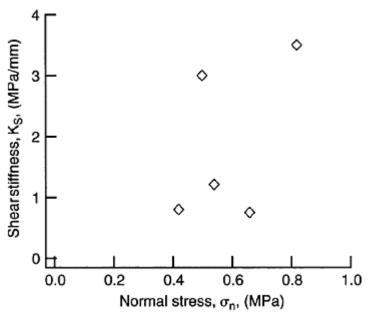


Figure 8d. Shear stiffness vs. normal stress, *JRC*=9–13.

between shotcrete and rock asperities. Remnants of cleanly sheared shotcrete asperities, often glued to the rock surface, were occasionally found within the rock valleys.

- 5. Ripping through natural flaws in both trachyte and magnetite were significant in the cases where steel fibres penetrated the flaws.
- 6. The effects of steel fibres were clearly marked by considerable scratching, peeling and often plugging of rock surfaces, which appeared to depend on rock surface hardness and inclination of the steel fibres with respect to the sliding plane. The steel fibres angled vertical to subvertical in the direction of sliding seem to have the most pronounced effect.

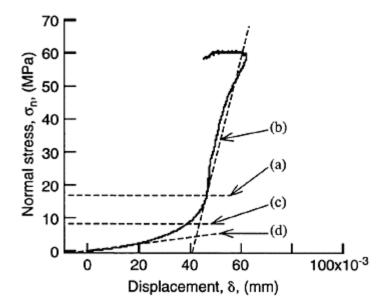


Figure 9. A typical joint compression curve resulting from compression of shotcrete-rock joints. (a) Joint compressive strength (*JCS*). The joint is assumed fully closed at this point. (b) Linearity indicating compression of intact material. (c) 50% of *JCS*, tangent at this point on stressdisplacement gives K_{n50} (or stiffness at 50% *JCS*). (d) gradient of this line gives K_{ni} (initial normal stiffness).

4.2 Joint tensile test results

The stress at which the shotcrete-rock interface came apart in tension was considered as the adhesion strength of the interface. Of the 7 samples tested average adhesion strength of 0.58 MPa was obtained. The joint tensile stiffness, $K_{t(50)}$, determined from tangents at 50% of the peak stress on the stress-displacement plot is 248 MPa/mm. Also the initial tensile stiffness, K_{ti} , determined from the linear portion at the origin is 293 MPa/mm. The values are close and have the same order of magnitude.

4.3 Joint compression test

The average joint compressive strength (JCS) obtained from testing 4 shotcrete-rock joints (2 shotcretetrachyte and 2 shotcrete-magnetite) is 16.0 MPa. The JCS was

determined as the stress corresponding to maximum joint closure, see Figure 9. The average joint normal stiffness, $K_{n(50)}$, determined from the tangent at 50% of the *JCS* is 288 MPa/mm for the 4 samples tested. Since the normal load being concerned with in study is less than 1.0 MPa, the initial joint normal stiffness, K_{ni} , appeared to give a good approximation of joint normal stiffness under such normal loads. For each joint the K_{ni} values were determined from the gradients represented by line (d) in Figure 9. Averaging for the 4 samples tested resulted in an average K_{ni} value of 100 MPa/mm.

Parameter	Value for <i>JRC</i> =1–3	Value for <i>JRC</i> =9–13
Joint bond strength (MPa)	0.51	1.37
Joint friction: ϕ_p	39.7°	47.4°
φ _r	35.4°	39.0°
Joint compressive strength (MPa)	16.0	_
Joint adhesion strength (MPa)	0.58	_
Joint compression stiffness:		
K _{ni} , (MPa/mm)	100	_
<i>K</i> _{<i>n50</i>} , (MPa/mm)	288	_
Joint tensile stiffness:		
<i>K_{ti}</i> (MPa/mm)	292	_
K_{t50} (MPa/mm)	248	-
Joint shear stiffness:		
K _s , (MPa/mm)	1.0	2.0

Table 4. Summary of shotcrete-rock joint strength properties.

4.4 Shotcrete compression test results

The average uniaxial compressive strength of shotcrete was 56.3 MPa. Average age of shotcrete was 50 days.

5 DISCUSSIONS

To assist in the discussion of shotcrete-rock interface strength and its mechanical properties the test results are summarized in Table 4. The tensile and compressive strength parameters for interfaces having *JRC* values of 1 to 3 are reported.

For most practical cases where shotcrete is used with rock bolts the normal load on shotcrete lining seldom exceeds 0.2 to 0.5 MPa. For such normal loads the shear strength

is seen to be determined by bond strength, which is essentially the cohesive strength of the interface. From Table 4 it can be seen that the average bond strength of the interfaces with JRC of 9 to 13 is more than 2.5 times the average bond strength of the interfaces with JRC of 1 to 3. The high average bond strength observed for interfaces with JRC of 9 to 13 is believed to be attributed to the failure mechanism that occurred when the peak strengths were attained. It is believed that a simultaneous failure of the bond and the shotcrete asperities may have resulted in the high average bond strength. After-test surface examination revealed cleanly sheared shotcrete asperities within the rock valleys, which appeared to have been sheared off simultaneous with bond failure. Quite often some of these failed asperities were still cemented to rock surface within the rock valleys without being crushed during sliding. The consequence of shotcrete asperities failing simultaneously with bond failure was that the shotcrete more or less had few or no asperities at all when sliding began.

The above complex failure mechanism was not seen or was negligible for interfaces with *JRC* values of 1 to 3. This was probably why the magnitude of the bond strength appeared to be close to the magnitude of the adhesion strength determined from tensile test for this *JRC* range.

Although the shear strength at normal stress greater than 1.0 MPa may not have any practical significance the implication is clear. For these normal stresses it is seen that the shear strength is markedly dominated by friction, resulting in cohesion being less significant. As seen in Figure 7a, this results in an impression that the peak shear strengths could easily be approximated with a straight line, thus masking the significance of cohesion unless separated. Perhaps the most serious consequence is that the true cohesion can be considerably underestimated.

Friction angles are some percentage higher than the friction angles reported by Barton (1988) for rock-rock joints at Kiirunavaara mine. Perhaps this is attributed to the considerable contribution by the steel fibres to the frictional component. After-test surface examination revealed considerable amount of surface traction caused by steel fibres during sliding, which were more pronounced for interfaces having higher *JRC* values. In some cases the steel fibres implanted inside rock flaws caused occasional localized tensile ripping. The different intensities of surface damage in terms of scratching and peeling, caused by steel fibres, on the two rock types are probably attributed to the hardness of the rocks and cleavage (especially in magnetite). Future work may be required to investigate the role of steel fibres during shotcrete-rock interaction.

It was necessary to measure the values of normal stiffness (i.e. for K_n and K_t) at normal stresses that are observed in practical cases when shotcrete is used with rock bolts. Hence, it appeared that the initial and average stiffness (measured as K_{ni} or K_{ti} for initial stiffness and K_{n50} or K_{t50} for average stiffness) would give practical values, whereas the stiffness at the maximum stress required to fully close (in compression) or separate (in tension) the joint would result in stiffness values at normal stresses which do not represent practical cases. Conversely the shear stiffness or K_s values were determined directly since the normal stresses at which the peak strengths were attained corresponded to those observed in practical cases. In this study the K_s values are essentially the stiffness of the bond without the frictional component. The K_s for interfaces with JRC of 9 to 13 is the combined stiffness of the bond and the shotcrete asperities, where as for interfaces with JRC of 1 to 3 it is only the stiffness of the bond. This is the reason for the difference in the K_s values for the two interface types. For the average normal stiffnesses, K_{n50} and K_{t50} , the values are in principal the same.

In measuring the joint displacement in tensile and compression, it was important to also measure the deformations on the whole specimen for sensitivity and surety that the CODs were only responding to the joint closure and not any deformation in the general body of the specimen or the intact material. The joint displacement curves were clearly distinct from the whole specimen deformation curve and therefore the joint displacements are considered reasonably accurate.

The compressive strength of the shotcrete-rock interface, i.e. JCS, was found to be approximately 27% of the compressive strength of shotcrete, which was the weaker half of the test specimens. Thus it can be assumed the JCS of shotcrete/rock interfaces is within 25 to 30% of shotcrete's compressive strength, which was 56.2 MPa.

Several difficulties were encountered during the observation and analysis of the results, chiefly because of the complex interaction and failure mechanisms involved. The first of these difficulties came about when attempting to separate shear strength corresponding to bond strength from those corresponding to combined bond strength and frictional effects. Audible and visual observations were keys to some of the decisions made. To keep the observations simple they were basically categorized into one of two typical and clear-cut cases shown and discussed in section 4.1.1. Observations that involved concurrent tensile ripping through the rock flaws caused by steel fibres, with no clear indication of shearing were omitted from the evaluation to avoid introducing error, which could have resulted in incorrect values for the parameters. The next difficulty was encountered in the observation and analysis of interfaces with *JRC* of 9 to 13. Here, even the bond failure was complex partly because of the concurrent bond and shotcrete asperities failure. Failure mechanisms were more complex for this case because of the increased surface roughness. Clearly defining the secondary peaks or in some cases residual stresses were made difficult by the complex failure mechanisms.

6 CONCLUSION

The results show that the bond strength is important for shotcrete-rock interfaces even when the prevailing failure mode is by shearing. This was seen to be particularly true for the normal loads generally observed on shotcrete linings in practical cases. For higher normal stress (σ_n >1.00 MPa) it is the frictional component, which is significant. Since friction was seen to be more dominating than cohesion in this normal load range there is a risk of underestimating the cohesion quite significantly. Steel fibres have been observed to contribute considerably to the frictional component.

The bond strength value (from direct shear test) was seen to be fairly close to the adhesion strength value (from tensile shear test) when no simultaneous shotcrete asperities failure occurred with bond failure. Interfaces with high *JRC* values have a tendency to produce higher bond strength because of the concurrent failure of the bond and the shotcrete asperities.

The average normal stiffness values, K_{n50} and K_{t50} , obtained from joint compression and tension were found to be in principal the same. However, the shear stiffness, K_s , largely depended on the failure mechanism. If the bond failed cleanly, i.e. without concurrent asperities failure then the shear stiffness is the stiffness of the bond. Conversely if the bond failure was the result of a simultaneous bond and shotcrete asperities failure then the stiffness is the inherent cumulative stiffness of the bond and the shotcrete asperities. Interfaces with high *JRC* values have the tendency to fall in this category of stiffness.

The laboratory tests, particularly the direct shear tests, clearly demonstrated the complex interaction and failure mechanisms that take place when shotcrete interacts with rock. The effect of steel fibres within the shotcrete was quite obvious. Apart from contributing to friction through surface traction it created an additional failure mode in the form of tensile ripping due to steel fibres implanted inside the natural flaws in the rock. In this case tensile failure was observed to occur simultaneously with shearing and sometimes this led to difficulty in identifying whether the peak strength corresponded to shear failure or the tensile failure. Despite this and other difficulties that arose as a result of attempting to simulate field conditions as closely as possible the results and observations give reasonable and perhaps reliable appreciation of the strength parameters of the interface and interaction as a whole.

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Investigations into mechanisms of rock support provided by sprayed liners

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ABSTRACT: Shotcrete has been used for rock support for many years, and the use of thin spray-on liners for this purpose has been increasing in recent years. The actual behaviour of such liners in their supporting mode is not well understood. In an attempt to provide some contribution to the knowledge in this regard, the paper will discuss a range of mechanisms of rock support provided by sprayed liners ("thin" shotcrete layers and thin spray-on liners) as background to the description of the results of a series of analyses of the support action provided by such liners. The relative contribution to the support action provided by the following liner parameters will be dealt with: liner thickness, liner stiffness or deformation modulus, liner tensile strength, liner-rock bond strength, and liner penetration into joints or fractures.

1 INTRODUCTION

Shotcrete has been used for rock support in tunnels for many years, and is known to be a very satisfactory means of support. Traditionally, the design of shotcrete support has been based on the assumption that the shotcrete layer acts as a structural arch (similar to a thin cast concrete lining) or as a series of beam elements acting between point supports provided by rockbolts or cables. Whilst this design approach may be acceptable in civil engineering practice, where conservatism is necessary to ensure long term stability, it is not appropriate in a mining environment in which stability margins must usually be much less conservative. In fact, in such an environment, some instability should occur to prove that mining is being carried out economically. Such instability should not occur unexpectedly, however, nor result in unsafe conditions.

In mining excavations, thin applications of shotcrete are common, and, according to a structural arch assumption, the support action would be theoretically negligible.

However, it is known that thin layers of shotcrete often do provide effective support, and a comment has been heard to the effect that, "the rock starts to behave when it sees the shotcrete." The mechanisms of support behaviour of the shotcrete must therefore be very different from the theoretically assumed structural arch or structural beam behaviour, and there are no established methods of support design in this case.

The importance of mechanisms of liner behaviour is even more acute in the case of thin spray-on liners (TSLs), whose use in the mining industry is on the increase. These liners are sprayed onto the rock surface with a typical thickness of about 4 mm. They are usually very flexible in comparison with shotcrete and their structural capacity is therefore negligible. Nevertheless, their supporting performance is often reported as being better than the expectations.

In an attempt to address some of the issues regarding support using thin spray-on liners (including thin layers of shotcrete), a programme of numerical modelling has been carried out, using the Phase² finite element analysis program, in which several influences on liner support action have been investigated. Important results from these analyses are presented below.

2 MECHANISMS OF THIN LINER SUPPORT

A range of mechanisms of surface support behaviour, and loading behaviour, of liners has been described by Stacey (2001) and by Tannant (2001). These support mechanisms might occur individually and in combination, and those that are considered to be particularly relevant to thin (say less than 20 mm thick) liners are summarised as follows:

- promotion of block interlock: the effect of this mechanism is the preservation of the rock mass in a substantially unloosened condition. There are several sub-mechanisms involved in the promotion of block interlock;
- the interlock that is promoted by the bonding of the liner to the rock, and the tensile strength of the liner, preventing shear on the interface and restricting block rotation;
- the development of shear strength on the interface between the liner and the rock as a result of irregularity of the interface surface. This mechanism is dealt with in some detail by Windsor (1998), but its effect is expected to be small in the case of thin liners, and it is not dealt with in this paper;
- the penetration of liner material into joints and cracks, which will inhibit movement of blocks. This is applicable to all jointed rock situations, including very high stress situations in which some loosening and stress fracturing will have taken place;
- prevention of block displacement by two mechanisms—the shear strength of a stiff liner, and the tensile strength of a thin bonded liner.
- air tightness: for a rock mass to fail, dilation must take place, with opening up occurring on joints and fractures. If such dilation can be prevented, failure will be inhibited. Coates (1970) suggested that, if the applied surface support is air tight, entry of air will be prevented or limited, and hence dilation will be restricted. This mechanism is identified as a contributory support mechanism by Finn et al (1999). Although this mechanism is unlikely to be effective in a static loading environment, in dynamic loading situations, in which rapid entry of air into the rock mass will be restricted, it is possible that an air tight liner might promote stability.

- basket mechanism: when the surface support develops the form of a basket, which then contains the failed rock, it will be acting mainly in tension. In this situation there are two considerations: firstly, the flexural rigidity or ductility of the liner, which will serve to resist the deflection of the liner to form a basket; secondly, the tensile strength of the liner itself.
- slab enhancement: slabs or incipient rock slabs, formed under high stress conditions, may fail due to buckling. The application of surface support effectively decreases the slenderness of the slab and increases its buckling resistance.
- beam enhancement: this is similar to slab enhancement—surface support on the underside of a roof beam may enhance the bending performance, and hence stability, of a roof beam.
- extended "faceplate": all surface support will extend the area of influence of rockbolt and cable faceplates or bearing plates, the effect likely to be greater for stiffer liners.

Localized deformation of liners may lead to localized failure. Therefore, even if the liner material has an elongation of 100% or 200%, the localization of deformation may result in failure after a total deformation of only a few millimetres. In such cases, it may be preferable for the bond between surface support and rock to be less effective, to allow some shear to take place on the interface, and hence for the deformation to be less localized.

Surface support is usually only one component of a support system, the common complementary support being rockbolts. The interaction between the surface liner and the rockbolts is extremely important. The behaviour of the rockbolts influences the behaviour of the surface support and may dictate the characteristics desired of this support.

3 INVESTIGATION OF MECHANISMS OF LINER SUPPORT

As indicated above, there are no established support design methods for thin liners. A reason for this may be that the magnitudes of the contributions, and the relative contributions, of the different mechanisms of support are unknown. In an attempt to remedy this lack of knowledge to some extent, a programme of analyses has been carried out to investigate the influence of several parameters on the support capacity. These are:

- the thickness of the liner;
- the stiffness (deformation modulus) of the liner;
- the tensile strength of the liner;
- the bond strength between liner and rock;
- the penetration of liner material into a joint or fracture.

Some of this work is a duplication of that carried out by Kuijpers and Toper (2003), but using an alternative approach. The aim of the analyses was to determine a comparative set of results that could give an indication of the relative importance of the different parameters. The support capacity of the liner was determined for the point at which complete failure of the liner was indicated by the analysis. Although a limited amount of localised shear failure occurred, ultimate "collapse" was due to tensile failure of the liner in all cases. In the results presented below, absolute values of support capacity are not indicated. The reason for this is that the overall stability (or the point at which failure is indicated), and hence the absolute magnitudes of load at which support fails, will depend significantly on the overall deformational behaviour of the rock mass, and particularly on the shear strength behaviour of joints in the rock mass.

Each of the parameters will be dealt with in turn in the sections below.

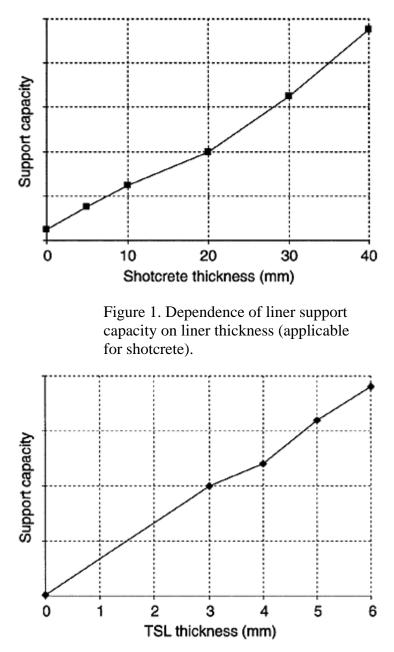


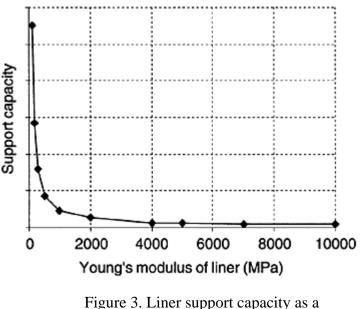
Figure 2. Dependence of liner support capacity on liner thickness (applicable for a TSL).

3.1 Liner thickness

It is logical to expect that, the thicker the liner, the greater will be its support capacity. This was confirmed by the results of the finite element analyses, which showed effectively a linear load capacity-thickness relationship, as shown in Figure 1. This figure is applicable for a shotcrete lining. For a thin spray-on liner, in which the thickness is much less, the result is also a linear relationship, as shown in Figure 2. It is probable that the deviation from linearity in these two figures is due to numerical effects and the difficulty of interpreting precisely the point at which "failure" occurs.

3.2 Liner stiffness

The greater the stiffness of the liner, the lower the support capacity provided by the liner. This result from the modelling is illustrated in Figure 3, and is similar



function of liner stiffness.

to the findings of Kuijpers and Toper (2003). The reason for this behaviour is believed to be greater localised stress concentration in the stiffer liner as a result of localised bending of the liner. The stiffer liner attracts stress, whereas the more flexible liner deforms more easily and therefore does not concentrate the stresses. As indicated by Kuijpers and Toper

(2003), the stiffer liner will also enhance the extent of liner debonding, and this will contribute to a greater bending moment, and hence stress, in the liner.

It should be noted that a stiff liner will be most effective in inhibiting the onset of rock movement. Once rock movement has initiated, however, it may be concluded from these results that a flexible liner will be more beneficial for support capacity than a stiff liner. A parallel in this case may be wire mesh rock support—it is very flexible and does not inhibit initial rock movements, but usually provides excellent rock containment support.

3.3 Liner tensile strength

The modelled relationship between support capacity and shotcrete liner tensile strength is shown in Figure 4. This illustrates that the relationship is effectively linear for tensile strengths up to about 5 MPa, and this range is representative for shotcrete and TSLs.

3.4 Liner shear strength

Since failure of the liner support was in the tensile mode in all cases, it might logically be expected that liner shear strength would have little effect on liner support capacity. The results of the finite element analyses showed that variation in the frictional strength of the liner material has no significant effect on the support capacity. Similarly, particularly for more flexible liners, the cohesive strength of the liner has little effect

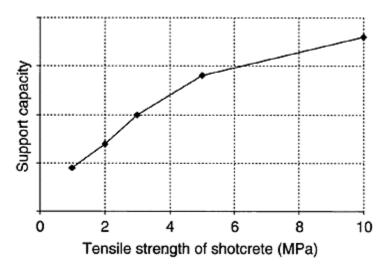


Figure 4. Liner support capacity as a function of liner tensile strength.

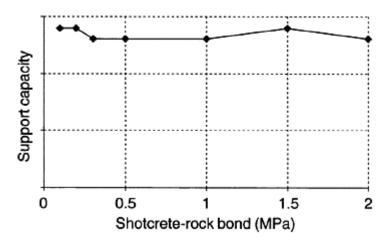
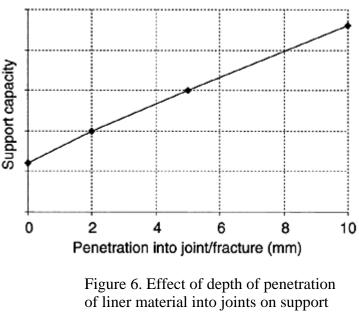


Figure 5. Effect of rock-liner bond strength on liner support capacity.

on liner support capacity. Should the cohesive strength of the liner be significantly lower than the tensile strength for any reason, failure of the liner will be due to shear rather than tension, and the above comments will then be invalid. The conclusions regarding the effect of liner tensile strength will correspondingly also be invalid.

3.5 Liner-rock bond strength

The bond between liners and the rock has been identified as being important with regard to the support performance of the liner (Kuchta et al, 2003 (referring to Holmgren), Malmgren et al, 2004). Perhaps unexpectedly, therefore, the results of the modelling have shown that the bond strength between the liner and the rock has no significant influence on the support capacity provided by the liner. This is illustrated in Figure 5, typically for shotcrete conditions. Based on published results of typical TSL and shotcrete bond strength tests, the bond strength range in Figure 5 covers the most likely values.



capacity.

As with the effect of the stiffness of the liner material, it is expected that good rock-liner bond strength will assist in inhibiting initial rock movements by various mechanisms. This effect has not been investigated here.

3.6 Joint penetration by liner material

As indicated above, penetration of liner material into joints and cracks will inhibit rotational movement of blocks. The analyses carried out did not deal with this rotational movement. The liner material will also bond blocks together and inhibit shear between adjacent blocks. Figure 6 illustrates the improvement in support capacity, determined from the modelling, as a function of the depth of penetration of liner material into joints or fractures. It is clear from this figure that liner penetration into joints and fractures is a significant support mechanism.

4 RELATIVE CONTRIBUTIONS OF LINER SUPPORT MECHANISMS

From the results of the programme of finite element analyses described above, it is possible to identify the relative contributions of different liner support mechanisms:

- The results show that, the stiffer the liner, the lower the support capacity. This corresponds with the results of pull tests on shotcrete and TSL panels described by Tannant (2001), which showed that peak strengths for shotcrete were attained at much lower relative displacements than for the TSL. The implication, perhaps, is that a

flexible liner will continue to promote rock block interaction, and therefore support, for a substantial time after movements have initiated, whereas a stiff liner that fails after limited rock movement will provide much less overall support capacity.

- The bond strength between liner and rock shows no influence on the support capacity. This conclusion is applicable for a stiff or a flexible liner.
- The thickness of the liner has a very significant influence on the support capacity. It is therefore considered useful to express the contribution of the other parameters tensile strength and penetration of liner material into joints—in terms of thickness of liner.
- An increase in liner tensile strength of 2 MPa is equivalent to an increase in shotcrete thickness of about 2 mm, or about 4% of a typical 50 mm shotcrete layer. For a TSL, the equivalent increase in thickness is about 9 mm, or more than 200% of a typical 4 mm thick layer. The conclusion is that an increase in tensile strength is very beneficial for support capacity in the case of a TSL.
- Penetration of shotcrete to a depth of 10 mm into a joint or fracture is equivalent to an increase in shotcrete thickness of nearly 4 mm, or about 8% of a typical 50 mm thick layer. The same depth of penetration of a TSL is equivalent to an increase in TSL thickness of about 0.7 mm or approximately 17% of a typical 4 mm thick layer.

For a stiff liner such as shotcrete, the influences of the parameters, in descending order of influence are thickness, penetration and tensile strength. For a flexible TSL, the corresponding order is tensile strength, thickness and penetration.

5 CONCLUSIONS

The following conclusions can be drawn from the research into the support provided by liners described in this paper:

- (i) liner support capacity increases linearly with liner thickness;
- (ii) penetration of liner material into joints and fractures constitutes a significant support mechanism provided by liners;
- (iii) better support capacity is provided by a flexible liner. The stiffer the liner, the smaller the movements that it can withstand before failing;
- (iv) rock-liner bond strength has no significant effect on liner support capacity;
- (v) with regard to (iii) and (iv) above, it is expected that a stiffer liner and a higher rockliner bond strength will both contribute to the inhibiting of initial rock movements, which will enhance stability. This was not investigated. Once movements have taken place, conclusions (iii) and (iv) become valid.

As a final statement, it must be noted that several of the identified liner support mechanisms have not been taken into account in the analyses reported in this paper. One of these, that of mechanical/shear interlock due to irregularity of the excavation surface, is probably important, but no indication can be given of its relative contribution compared with the other parameters investigated in this paper.

ACKNOWLEDGEMENT

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Large scale static laboratory tests of different support systems

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ABSTRACT: This paper presents the results of tests on 1.5×1.5 m panels subjected to static load under conditions simulating field loading and supported with rock bolts on a 1×1 m pattern. The panels were made up of diamond wire mesh (chain link) or using reinforced shotcrete. Shotcrete reinforcement was either a single layer of diamond mesh or synthetic fiber. The load over the diamond mesh panels was increased up to a limiting central displacement of 20 to 25 cm. Average thickness of the mesh-reinforced-shotcrete panels was 7.9 cm; the average yield load was 113 kN at a yield displacement of 6 cm. The behavior of the synthetic fiber-reinforced panels was dependant on the fiber content and thickness of the shotcrete. The yield strength varied from 118 up to 294 kN and the yield displacement from 3.7 cm (for the strongest panel) up to 7 cm. The field behavior of the different linings is also presented.

1 INTRODUCTION

Usually, underground support systems are made up of different components, for example rock bolts or cables can be used as reinforcing elements and shotcrete, woven or welded wire mesh, or mesh and lacing can be used as the retaining elements. Due to the difficulty of carrying out a numerical analysis of such support systems, several testing procedures have been implemented to determine their behavior under either static or dynamic loading.

Little (1985) carried static load tests on 240×240 cm panels of shotcrete reinforced with either steel fiber (576 N/m³ of 30×0.5 mm Dramix steel fibers with deformed ends) or with wire fabric (100 mm square 8 gauge welded wire fabric). The panels were anchored with four rock bolts spaced at 2 m and tensioned to 130 kN. The load was applied vertically upwards over a 10 cm-diameter steel plate at the center of the panel.

The central load on 11.5 cm thick steel fiber reinforced panels reached approximately 50 kN at 2 to 10 mm displacement and reduced to 30 kN at 30 mm displacement, remaining approximately constant up to 70 to 110 mm of vertical displacement.

Kirsten (1993) reports the results of static loading tests on 160×160 cm diamond mesh and steel fiber-reinforced shotcrete panels. The reinforcement mesh was made up of 75 mm aperture and 3.1 mm diameter strand placed in the middle half of the panels. A uniform load was applied vertically upwards and the sides of the panel were restricted to move in the vertical direction. In some panels the load was applied vertically upwards over a 10 cm square plate at the center of the panel. In other tests, a uniform load was applied in the central portion of the panel by means of a hydraulically pressurized bag. In a typical test of a uniformly loaded 10-cm thick mesh reinforced shotcrete panel supported with 4 rock bolts at 1-m spacing, the maximum load capacity of 150 kN was reached at approximately 25 mm of displacement, decreasing steadily to 75 kN up to approximately 200 mm; it was assumed zero after 250 mm.

The results of Little and Kirsten are difficult to compare because the dimensions of the panels, the spacing of the rock bolts and the boundary conditions were different between the two series of tests. Both testing programs used static loading. For the study of rockburst phenomena it is important to know the dynamic response of the support system.

Tannant et al. (1995) report the results of static and impact tests on welded wire mesh (#6 gauge mesh, 4.88 mm diameter and 10.3 kN maximum strength) and on shotcrete reinforced with welded wire mesh. The panels were restrained by bolts and plates located on the top of eight columns positioned so as to model a symmetric 1.2×1.2 m diamond rock bolt pattern. Pulling a 0.6 m diameter plate with rounded corners simulated the static rock loading and dropping a 565 kg and 0.6 m diameter cylindrical drop-weight from a prescribed height directly on the panel simulated the dynamic loading. No other side restriction other than that provided by the rock bolts was used in the tests. The results of the static and dynamic tests could not be compared in a simple manner because of the unknown amount of the kinetic energy dissipated within the support system by stress transmission or other effects. Nevertheless, the results suggest that, at the same panel deformation of 25 cm, the reinforced shotcrete was able to absorb significantly more energy during the impact tests (16 kJ) compared to the static tests (9 kJ). The static and dynamic energy versus displacement behavior of the mesh were similar, but the total displacement under impact load was much larger (up to 60 cm) than in the static loading tests (up to 30 cm).

The dynamic testing facility developed by Ortlepp and Stacey (1997) differs substantially from the one used by Tannant et al. (1995). The main differences refer to the boundary conditions. Lateral restriction of the mesh (to prevent the movement on its own plane) was provided by a steel frame connected to ground anchors via wire ropes. A simulated rock mass consisting of rock blocks was laid over the test panel and the impact of a falling drop weight was distributed onto the simulated rock mass by a distribution pyramid of steel cylinders. The tests results show little difference in the energy absorption capacity of weld mesh reinforced shotcrete and diamond wire mesh.

Concerning the boundary conditions, Thompson et al. (1999) point that the real restraint beyond the line of bolts is largely unknown as it depends on the location of the loading relative to the bolts and the edges of the mesh. Their test on welded wire mesh

without lateral restraint show that the behavior is strongly dependant, among other factors, on the orientation of the mesh relative to the bolt pattern.

The tests reported herein show that the connection of two meshes is certainly a weak link in the support.

2 TESTING PROCEDURES

2.1 Equipment

The panels reported in this paper were subjected to a flexural load trying to reproduce the boundary and loading conditions that may be encountered in a tunnel.

The test set up, shown in Figure 1, follows the recommendations given by Prof. D.Stacey. The panel is supported at four points with $100 \times 100 \times 10$ mm steel plates and rock bolts spaced at 1×1 m. To simulate the load of broken rock, the load of a hydraulic jack is applied vertically downwards and distributed on the panel via a pyramid of steel cylinders and concrete blocks. In order to reproduce the displacement restriction imposed by the ground, a concrete ring tightly adjusted with a cement grout restrains the

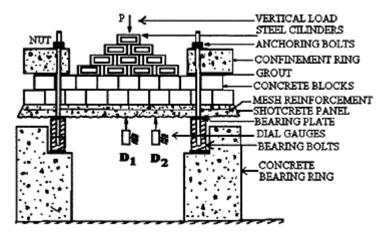


Figure 1. Test set up.

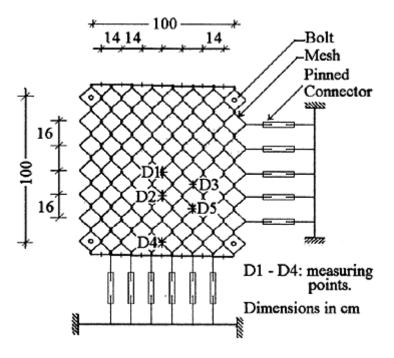


Figure 2. Boundary conditions for testing mesh panels.

upward displacement of the panel as well as its rotation at the anchoring points. The sides of the panel are free to move downwards for approximately 15 cm. The test arrangement for wire mesh panels is similar, but pinned rods provide lateral restriction connected to a stiff beam, see Figure 2. A temporary support allows maintaining the mesh horizontal while placing the concrete blocks and steel cylinders. The mesh displacement due to the self-weight of the simulated rock mass is measured as the temporary support is lowered at the beginning of a test.

2.2 Tests of diamond (chain link) mesh

The tests were run on samples of mesh provided by two different manufacturers. The geometrical and mechanical properties of both meshes (A and B) are given in Table 1. Mesh B was slightly thicker than mesh A because the radius of curvature of its lacing was slightly larger than that of mesh A.

Figure 3 shows the average load-displacement and energy-displacement behavior of both types of mesh.

Mesh type	Properties of the wire			L×H (cm)	N
	φ(mm)	P _u (kN)	$\epsilon_u(\%)$		
А	5.11	9.68	12.0	14×16	6
В	5.07	10.32	10.5	14.9×14.3	4

Table 1. Physical characteristics of the meshes.

Notes: $\phi_{=\text{diameter}}$

P_u=Rupture load

 ε_u =rupture strain

L=length of one diagonal of the "diamond"

H=length of the other diagonal

N=number of panels tested.

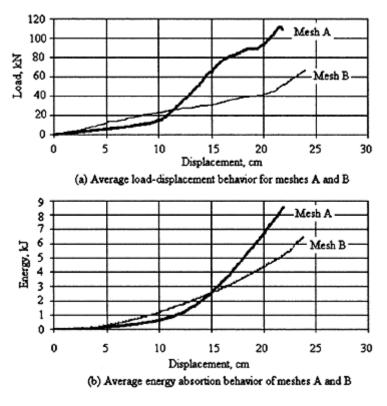


Figure 3. Laboratory behavior of diamond meshes.

Typical standard deviation on the load is ± 10 kN. Some panels were loaded up to a displacement of 32 cm but the curves are drawn up to the displacement for which there is

data for all the panels tested. Failure of a wire was observed in only one panel, of mesh A, at approximately 90 kN of load. Failure took place at the edge of the steel plate. Figure 3 suggests that both meshes yielded at a displacement of 20 cm and then their stiffness increases abruptly. The smaller stiffness of mesh B is probably related to the larger radius of curvature of the lacing. It is felt that such stiffness increase after a displacement of 20 cm is due to the arch effect produced by the concrete blocks and shown in Figure 4. Certainly, the arch effect should depend on the geometry of the blocks of broken rock. Because of the arch effect, the load increase is transmitted almost entirely to the bolts, which should then be able to start yielding in order to increase the energy absorption capacity of the complete system.

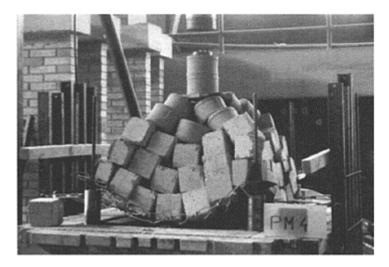


Figure 4. Arch effect after large displacements.

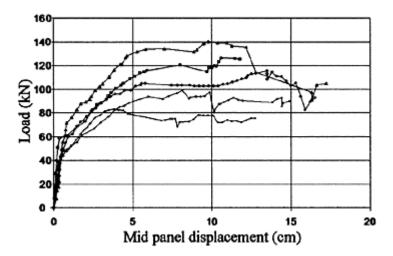


Figure 5. Results of tests on diamond mesh reinforced shotcrete.

A series of tests were carried out to study the performance of different ways of connecting the mesh. For that purpose, the connectors were installed along a central line of the test panel. Three type of connections were tested:

(i) A simple hook made with the end piece of the same wire of the mesh.

(ii) As in (i) plus a tie made with soft wire in order to prevent opening of the hook.

(iii) A hook made of 10 mm diameter construction steel.

The rupture load ranged from 83 up to 117 kN for connectors of the type (iii) down to 32 kN for type (i) connectors.

The above results emphasize the need for proper detailing during the design stage and of good construction control.

2.3 Tests of diamond mesh reinforced shotcrete

Figure 5 summarizes the results of tests on 5 panels of shotcrete reinforced with one layer of type A mesh.

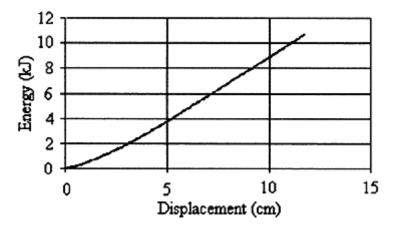


Figure 6. Energy absorption of mesh reinforced 10 cm-thick shotcrete panels.

The thickness of the panels ranged from 6.8 to 9.4 cm, with an average of 7.9 cm. The shotcrete strength, measured on four 51 mm diameter samples drilled from the panels and corrected for slenderness, ranged from 7.2 to 40.7 Mpa. Strong lamination existed in two out of six drilled samples. In most of the panels, some wires broke during the loading. Wire breakage was evident by the noise and the sudden change in the load-displacement curve (not shown in Figure 5).

Average values of some physical properties of the shotcrete panels are as follows:

- Initial stiffness=69 kN/cm
- Yield load=113 kN
- Yield displacement=6 cm

In some panels, an arch effect was noticeable at a displacement of approximately 10 cm. The arch effect was also reflected in an increase of stiffness of the load displacement curve.

Figure 6 shows the average energy absorption capacity measured in the laboratory tests.

2.4 Tests on synthetic fiber reinforced shotcrete

The tests on synthetic fiber reinforced shotcrete have been informed by Van Sint Jan et al. (2003). A short summary will be presented here.

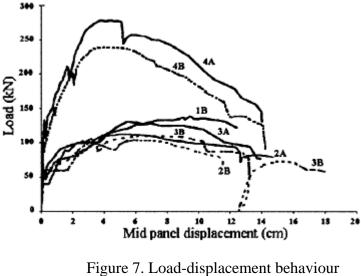
Six shotcrete panels, reinforced with Strux 85/50 fiber provided by the manufacturer, were prepared in the field and tested in the laboratory. An additional shotcrete panel reinforced with one layer of diamond mesh was tested for comparison. The physical properties of the panels are summarized in Table 2.

Figure 7 shows the total load as a function of the mid span deflection for all the panels. The load-displacement behavior was dependant on the panel thickness as well as on the type and amount of reinforcement. Panels 2A and 3A had the same thickness (11.9 cm), but an increase in fiber content from 58.8 to 66.2 N/m³ resulted in a strength increase from 127

Panel	Fiber (N/m ³)	Average thickness (cm)	Strength of shotcrete* (Mpa)
1B	Mesh	10,4	28,08
2A	58,84	11,9	26,00
2B	58,84	10,6	27,56
3A	66,19	11,9	18,72
3B	66,19	9,5	26,77
4A	88,26	13,5	17,68
4B	88,26	12,9	22,36

Table 2. Physical properties of the fiber reinforced panels.

*Diameter of the samples=49 mm.



of shotcrete panels reinforced with different amounts of synthetic fiber.

to 143 kN. The average thickness of the panels with 88.3 N/m³ fiber content was 12.9 and 13.5 cm and their yield load reached 251 and 294 kN respectively. Such strengths are 72% an 100% larger than the strength of the 10.4 cm-thick panel reinforced with chain link mesh and they suggest that the strength of a panel with 88.3 N/m³ fiber reinforcement would exceed that of a similar panel (of same thickness) with one layer of chain link mesh reinforcement.

The average initial stiffness of the fiber reinforced shotcrete panels was 59 ± 8 kN/cm. The initial stiffness of the mesh reinforced shotcrete panel was $k_i=38$ kN/cm.

Figure 8 shows the computation of the total energy input into the support system. The horizontal axis shows the displacement of the hydraulic jack loading point and the vertical axis the total applied load.

The results show that for a central panel displacement of up to approximately 10 cm the total energy absorption capacity of the fiber-reinforced shotcrete can be similar or even exceed the capacity of the mesh-reinforced shotcrete. Certainly, after a central panel deflection of 10 cm the shotcrete is fractured and needs to be repaired.

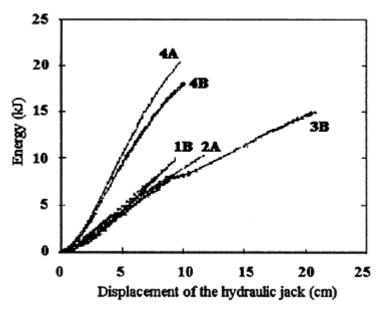


Figure 8. Comparison of energy absorption of the shotcrete panels reinforced with synthetic fiber and the mesh reinforced panel (1B).

3 FIELD OBSERVATIONS

3.1 Typical support systems used at El Teniente

The support elements used at El Teniente include steel bolts, nuts, plates, cables, mesh and shotcrete. All these elements have been standardized. The support system to be used in any particular sector takes into account, besides the geomechanical properties of the rock mass, the following restrictions:

- The useful life span of the opening.
- Transit of personnel or equipment.
- Relevance of the facilities with regards to the production process.

Rock bolts and mesh-reinforced shotcrete are used mainly in the roadways for personnel and equipments or those that are important for the production process. The support may be complemented with cables.

As shown in Figure 9, the steel plate of the rock bolt is placed over the mesh that is directly in contact with the rock surface; small cement blocks are used as separators in order to locate the mesh at the center of the 10 cm thick shotcrete layer.

Bolts and mesh are used in areas with low transit of people or equipments, or in areas with shorter useful life, or where the rock quality is better so that shotcrete is not deemed necessary.

Bolts and synthetic fiber reinforced shotcrete are being used in a test area away from dynamic loading.

A research program is being prepared to study the behavior of this support under dynamic loading.

3.2 Observed behavior

The onset of failure of the mesh-reinforced shotcrete, either under static or dynamic loads, is evidenced by the

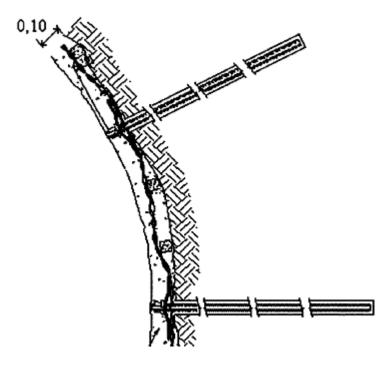


Figure 9. Typical layout of mesh reinforced shotcrete.



Figure 10. Initial failure of mesh reinforced shotcrete at 62 m of a magnitude 1.8 event.

cracking and swelling, as shown in Figure 10. Lining failure is often associated to the fall of a volume of rock that includes some bolts, the mesh unravels and some bolts are left in place without grout, as shown in Figure 11. Since the bearing plate is under the shotcrete there is no benefit of its (very small) punching shear strength.

Diamond mesh has shown high deformation capacity, particularly associated to dynamic loading. Mesh deformation is clearly evidenced by the deformation of



Figure 11. Complete failure of a wire mesh reinforced shotcrete at 82 of a magnitude 3.0 event.



Figure 12. Deformation of the mesh at 72 m of a magnitude 2.4 event.

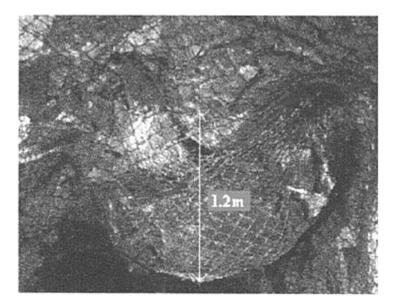


Figure 13. Large deformation of the wire mesh retaining broken rock at 166 m of a magnitude 2.2 event.

the rhombs as seen in Figure 12. The mesh is also capable to retain the broken rock, as evidenced in Figure 13. So far, there are no cases of failure in areas supported with bolts and synthetic fiber reinforced shotcrete.

Since the energy absorption capacity of the synthetic fiber reinforced shotcrete is similar to that of the diamond mesh reinforced shotcrete, a better use of the mesh might be obtained if it is placed outside of the fiber-reinforced shotcrete. According to our static tests, the mesh would start to contribute after the shotcrete begins to crack and the energy capacities could possibly be added. More analysis and tests are needed to support this hypothesis.

4 CONCLUSIONS

Laboratory tests and in situ observations suggest that the wire strands of the diamond mesh break more easily when used as reinforcement in the shotcrete than when they work alone.

Failure of the diamond mesh is critical because it tends to unravel.

The load-displacement behavior of mesh reinforced and fiber-reinforced shotcrete are similar.

The energy absorption capacity of a fiber reinforced shotcrete may be varied almost continuously and can be much larger than that of a shotcrete reinforced with a single layer of mesh. The laboratory tests suggest that if the arch effect is developed in the lining, then most of the ensuing load increase will go directly to the bolts that should be able to dissipate the energy input.

Failure of the mesh occurred in one test, at the edge of the bearing plate, suggesting that plate design is a critical issue.

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Soledad Celis, from El Teniente Division, participated actively during the development of this study and in the writing of chapter 3.

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The use of cementitious linings to protect ore passes in the mining industry

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ABSTRACT: When developing a new mine, development of the ore pass system is a major capital expense. To prolong the life of the ore pass system the mine needs to ensure that these ore passes are adequately protected when subjected to high stresses and adverse geological conditions. Various methods of attempting to prolong the life of these ore passes have been tried in the South African Mining industry with varying degrees of success. The most common method of lining ore passes today is by means of cementitious linings. Before developing an ore pass system the following need thorough investigation. The location of the ore pass, type of ore pass system to be used and the type of rock reinforcement to be used. The choice of the correct concrete as a lining material is critical (cement type and aggregate) to the durability of the lining. Laboratory results must be evaluated and compared to practical experience gained from actual installations when available. The method of application must be evaluated comparing the advantages and disadvantages of each method. Detailed cost evaluation of the various options must be done. Once this investigation has been completed the mine needs to carefully evaluate all the options available taking into account the cost to rehabilitate the ore pass if the ore pass system fails. Only once all these steps have been completed the mine should decide on which option to follow.

1 INTRODUCTION

In any underground mine getting the blasted ore from underground into the process plant is critical to the success of the operation. Depending on the nature of the deposit being mined and type of mining method employed a good ore pass system is essential to achieving this.

A mine's main ore pass system is expected to last until the mine closes but often design parameters used have not been investigated thoroughly enough and these ore

passes start failing after only a few years in operation. This results in disruption to production, high rehabilitation costs and often total abandonment of an ore pass system and redevelopment of new system.

It is not always possible for an ore pass to operate without any problems until the day the mine closes, the life of an ore pass can however be cost effectively optimized with the use of good design principles.

Good design principles must be based on theoretical, laboratory as well historical data.

2 DESIGNING A SUPPORT SYSTEM IN AN ORE PASS

According to Hagan and Acheampong (1999) the following factors must be considered when designing a support system in an ore pass.

Condition of the rock mass and presence of geological features.

Initial and anticipated state of stress.

Anticipated extent of failed rock.

Method of excavation, shape and inclination.

Purpose of ore pass and planned life.

Tonnage to be handled.

Strategic importance and time required for installation.

2.1 Condition of rock mass and state of stress

Ideally an ore pass should be located in a geologically stable area where little or no rock stress changes are anticipated. This however is not always the case.

In the South African gold mining industry most main ore pass systems are located within the shaft pillar which is ideal as this is a static stress environment but virgin stresses can be in the region of 90 Mpa in the Deep South African gold mines.

Figure 1 shows damage caused by stress to an ore pass located 3000 m underground in an area where weak rock was present.

Rock types in South African Gold Mines are generally competent, but not entirely homogeneous. Layers of quartzite of varying compositions are interbedded with weak shale bands and are transacted by occasional strong dyke and sill intrusive as indicated in figure 2 (An industry guide to methods of ameliorating the hazards of rock falls and rock bursts 1998). When an ore pass is located in such an area the possibility exists that the virgin stress could exceed the strength of the rock.

Apart from the inhomogeneties, other types of geological weakness transect the rock mass; these also play an important part when deciding on the location of an ore pass system. Parting planes, joints and minor

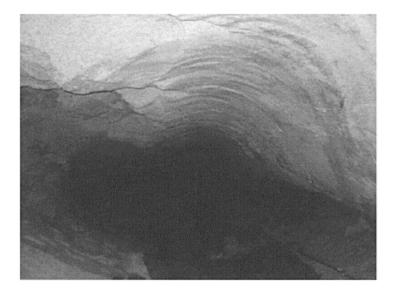


Figure 1. Rock failure around an unlined ore pass 3000 m below surface.

	UCS
 	(Mpa)
Quartzite (90%Q)	300
Quartzite (70%Q)	200
Shale	30-100
Lava	220
Dolomite	300
Dyke/sill	100-400

Figure 2. Simplified South African Mining rock properties (COMRO 1998).



Figure 3. View down an ore pass which has scaled to an average diameter of approximately 8.0 m.

faults need to be considered when designing an ore pass system. Major faults and dykes need to be avoided.

If the dip of the strata is favourable to the ore pass location the ore pass system should be developed at 90° or as close to 90° as possible to the strata.

A detailed geological analysis of a planned ore pass location can be very valuable in identifying any adverse conditions which can result in an ore pass problems at a later stage.

In the early stages of developing a mine the ore pass system can usually be moved to another location in close proximity to the original site without affecting the original mine design negatively if problems with the original location is anticipated.

2.2 Anticipated extent of failed rock

As with any excavation underground it is important to install the permanent support as soon as possible. Delay in this installation could result in unnecessary failure of the rock around the ore pass making installation of permanent support difficult resulting in high installed cost.

If an ore pass has already failed and has to be rehabilitated it is difficult to anticipate the exact extent of the failed rock and this extent usually can only be realized once the rehabilitation has begun. Figure 3 shows an ore pass which was not adequately supported and lined. The ore pass has now been resupported and is ready for lining.

2.3 Method of excavation, shape and inclination

The most common method of excavation of a main ore pass in the South African Mining industry is by means of raise boring. The raise bored ore pass will either be drilled to its required diameter and then supported and lined which is an advantage as no fractures

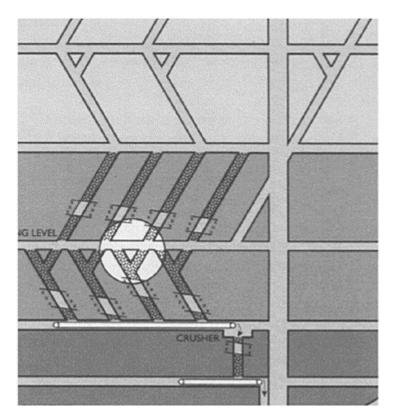


Figure 4. Typical layout of a South African Gold Mine ore pass system.

due to blasting exists, alternatively the original raise bored ore pass is slipped to the required diameter before supporting and lining. In the second case fractures due to blasting will be present.

Main ore passes in the South African Mining industry are usually circular in shape and inclination between 68° and vertical with 78° to 84° being the most common. Figure 4 shows the layout of a typical shaft ore pass system in a South African Gold Mine.

2.4 Strategic importance and time required for installation

The time required for an ore pass installation will vary from mine to mine. With a new mine there might be a reasonable delay before production starts and the pressure to get

the ore pass system into operation will not be as high as with a mine which is rehabilitating an ore pass system. Should the mine be operating a single pass system this rehabilitation must be completed as soon as possible to avoid unnecessary disruptions to production.

2.5 Type of rock re-inforcement

The most common method of supporting the rock walls of a main ore pass in South Africa is by means of grouted 16 mm rebar bolts usually around 1.80 m in length. This is combined with welded mesh if the ore pass is to be lined using a cementitious lining. Cementitious linings will be discussed in section three.

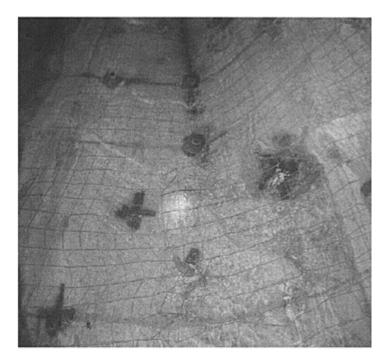


Figure 5. Typical rock support pattern in an ore pass on a South African Gold Mine.

Split sets are at times used as a temporary support before the permanent grouted rebar is installed.

Some mines have also incorporated 15.20 mm pre-tensioned fully grouted cable bolts into their ore pass support system. These cable bolts are usually around 4.0 m long.

Osae and De Lange (2003) have indicated the support resistance of a support system as described above to be 190 KN/m^2 before the application of the cement lining.

3 CEMENTITIOUS LININGS

Cementitious linings serve a dual purpose in an ore pass and this is to provide protection to the steel support members and secondly to protect the rock face against damage from the rocks which are tipped down the ore pass.

3.1 Cement types

Two types of cements are currently used as binders in concretes to line ore passes in the South African mining industry. These are Ordinary Portland Cements (OPC) and Calcium Aluminate Cements (CAC). Apart from the difference in the method of manufacture the two cements differ in chemical compositions as indicated in figure 6.

CAC also has certain properties which can offer the end user advantages when using CAC as a binder in ore pass lining concrete.

Concretes based on CAC have similar working times to those based on OPC but are rapid hardening which allows the return to service of installations

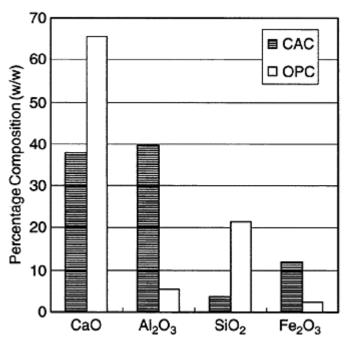


Figure 6. CAC/OPC chemistry comparison.

based on CAC concretes within 12 hours. This is ideal when repairing and ore pass which needs to be put back into operation relatively quickly.

CAC is also resistant to mild acids which are an advantage in areas where acidic ground water conditions are present.

CAC concrete is subject to a specific behaviour called "conversion". As a result of this process, the transient high early strength decreases to a lower but stable long term strength. Metastable "hexagonal" hydrates are initially produced by the hydration of CAC, over time these hydrates convert to stable cubic hydrates. The rate of this conversion is dependent on factors such as temperature, humidity, pH values in the pore fluid, and the presence of water (Robson 1962).

Today the technology is available to predict converted strengths of CAC concretes by means of laboratory tests; only the "converted" strength should be considered for design purposes. For mining applications like those presented hereafter, long term records demonstrate that CAC based concretes fit most purposes satisfactorily, even if conversion occurs over time.

3.2 Aggregate types

When choosing an aggregate for use in an abrasion resistant cementitious lining the aggregate has to be hard, tough and not fracture unfavourably when crushed so as to create weakness in the aggregates structure.

Various aggregate has been used in cementitious ore pass linings in the South African mining industry the best performing aggregate being alluvial Corundum $(A1_2O_3)$. Unfortunately alluvial Corundum is no longer available in South Africa. Corundum-Siliminite which used to be mined in the Northern Cape was also used but recent test results have indicated that a better performance is obtained when using Andesite and Alag® as aggregates.

At present Andesite is the preferred readily available aggregate used in the majority of concretes that are subject to abrasion and impact.

Alag[®] a synthetic aggregate when used with CAC and Andesite has proved to perform exceptionally well when used subjected to abrasion and impact.

3.3 Fibres and additives

In the South African Mining Industry fibres are now extensively used in concretes designed to resist abrasion and impact. Previously only steel fibres were used but over the past few years' synthetic fibres have proved not only to be more cost effective than steel fibres but also to outperform steel fibres when used in concretes which are subject to abrasion and impact.

For the purpose of this paper all other additions to the concretes referred to other than cement binder, aggregate and fibres will be referred to as additives. All of the above will be discussed in greater detail in section four.

4 EVALUATION OF THE CEMENTITIOUS LININGS AVAILABLE

In 2000 Placer Dome Western Areas JV decided to evaluate the various cementitious linings available on the market to determine which would be the most durable. This section deals with the first phase testing done by Parrish (2000).

4.1 Mix designs

The main groups tested were:

Supplier 1, CAC with various aggregates including
Andesite, Alag® and Corundum-Siliminite.
Supplier 2, OPC with chrome slag aggregate.
Supplier 3, OPC, silica fume, superfine fly ash,
Andesite or chrome aggregate, SRB type polymer and the latest super plasticizers.
The following sub-groups were also tested:
Various steel fibres at varying dosages.
Dry powder polymer for supplier 1 mixes.
Liquid polymer for supplier 3 mixes.
Plasticizer in supplier 1 and 3 mixes.
HPP poly fibre in various dosages.
Super fine fly ash in supplier 3 mixes.
Microfilament poly fibre.

4.2 Concrete preparation

All concretes were mixed using a 75 litre pan mixer.

The mixes were then cast into test panels and placed in a controlled climate room at a temperature of 35° C

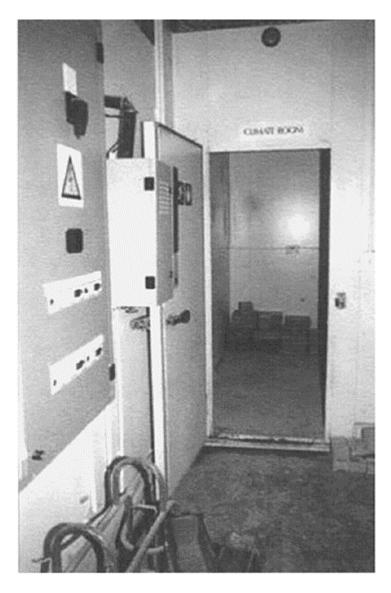


Figure 7. Climate room used to cure test panels.

and a relative humidity of 80% (typical conditions encountered in an ore pass in a South Africa deep level gold mine).

Specimens were kept in the climate room for 28 days but due to time constraints on the tumbling tester some specimens were kept for up to 56 days.

4.3 Testing method

Compressive strength (1, 3, 7, 28 and 56 day) as well as flexural strength tests (at 28 days) were done on all 41 mix designs tested.

All 41 mix designs were tested for abrasion and impact in a tumbling testing unit as illustrated in figure 8.

The test method used was a modified version of the SABS 541 test method in which four panels of each mix design are affixed around the internal periphery of drum.

The drum is loaded with 12.50 kg of 40 mm steel ball charges, illustrated in figure 9. The drum is then rotated for a period of 48 hours and the volume eroded on the test panels measured and recorded.

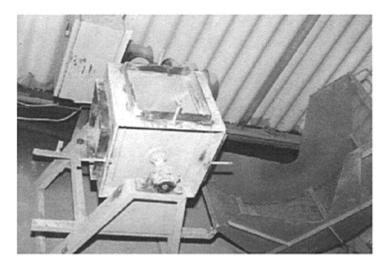


Figure 8. Tumbling testing unit.



Figure 9. 12.50 kg \times 40 mm steel ball charges.

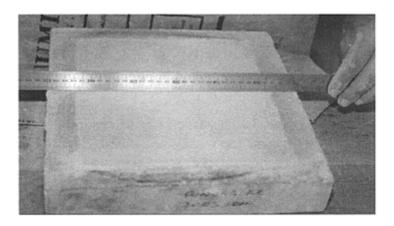


Figure 10. Measuring the volume of eroded material on the test panels.

4.4 Results

After 48 hours of tumbling 28 of the 41 mix designs had failed due to cracking or complete destruction.

All 13 of the mix designs which were not destroyed were fibre reinforced.

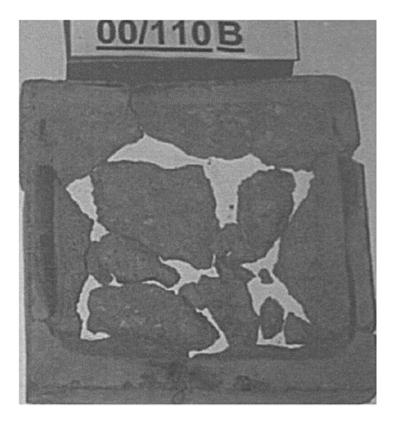


Figure 11. Destroyed OPC based test panel.

Of these 13 mix designs the top 6 were based on CAC.

The best CAC and OPC mixes contained polypropylene fibre.

Some of the OPC mixes were polymer modified.

Compressive strengths varied (79 to 119 Mpa).

Compressive strengths do have a part to play in creating durability but it is clear that it is not the main criterion when testing concretes for abrasion and impact resistance.

Flexural strengths showed less variation (7.0 to 8.8 Mpa).

The best CAC based concrete out performed the best OPC based concrete by a factor of 2.30 times.

4.5 Second phase testing

Mix designs used in the phase 1 testing programme by Parrish (2000) were improved where possible and tested using the same method as described in section 4.3.

Somers (2001) documented the results of the phase 2 test programme, the best improved CAC mix design now outperformed the best improved OPC mix design by a factor of 3.50 times.

Somers (2001) concludes that as illustrated in phase 1 of the testing programme there is no direct coloration between compressive strength and abrasion and impact resistance.

In the phase 2 testing programme all products out-performed the equivalent products tested in phase 1, CAC based concretes still proved more durable than OPC based concretes when subjected to abrasion and impact.

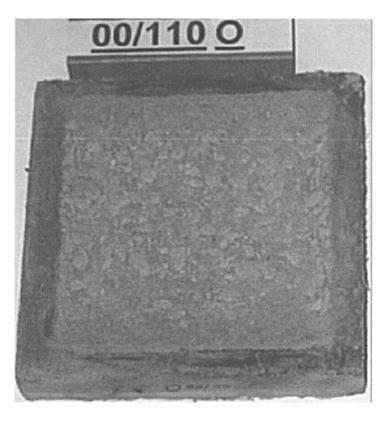
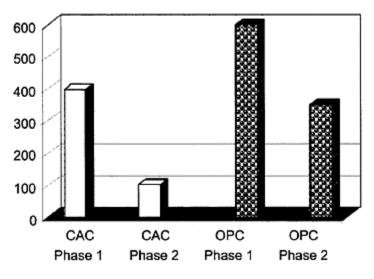


Figure 12. Test panel based on CAC, Alag[®], Andesite HHP Fibre and Fibrin.



Comparative volume loss @ 48 hours

Figure 13. Comparative volume loss best CAC based concrete vs. best OPC based concrete.

5 EVALUATION OF THE COSTS OF CEMENTITIOUS ORE PASS LININGS

All cost indicated in section 5 are based on raise boring an ore pass 100 m long with a diameter of 2.70 m.

Table 1 indicates that the total cost on an installation using CAC based concrete is 13.70% more expensive than the equivalent OPC based concrete. If the OPC based lining lasts for the entire life of the ore pass this is the option to use. However the period of the lining will last is near impossible to predict because of all the variables involved such as:

Those described in 2.1 previously as well as influences such as how often during the life of the ore pass it is allowed to run empty resulting in excessive impact damage caused by rocks falling great distances and subjecting the ore pass lining to unnecessary damage.

If the ore pass has to be rehabilitated at any time during its life span the CAC option now becomes more cost effective because of the high cost of rehabilitation.

Parrish (2000) indicated a life expectancy of 12 years for the best OPC mix design tested. The CAC mix designs will outperform this OPC product by at least a factor of 2 although second phase testing Somers (2001) indicates a factor of 3.50.

In South Africa the current inflation rate is around 5.0% per annum, for the purpose of the next table it is assumed that this remains so for the next 12 years. It is also assumed that the dimensions of the ore pass remains the same although this is never the case as an

ore pass which has to be rehabilitated has usually scaled to at least twice its original diameter.

The cost of using an OPC option now becomes 85% more expensive than if the mine had decided on a CAC option originally. This excludes any additional cost

Table 1. Initial cost of installing an ore pass lining in an ore pass with a diameter of 2.70 m and length of 100 m.

Item	Cost
Raise boring	R 2,500,000
Installation of support and lining	R 1,800,000
16.0 mm shepherd crooks	R 6,652
Cement capsules	R 4,384
Grouted cable anchors	R 11,500
Welded mesh	R 6,786
CAC lining 300 mm thickness	R 954,000
OPC based lining 300 mm thickness	R 318,000
Total CAC based	R 5,283,322
Total OPC based	R 4,647,322

Table 2. Cost to re-line a failed ore pass after 12 years in operation.

Item	Cost
Re-lining and supporting	R 4,490,000
16 mm shepherd crooks	R 11,973
Cement capsules	R 7,891
Grouted cable anchors	R 20,700
Welded mesh	R 12,216
OPC based lining 300 mm thickness	R 572,400
Total	R 5,115,181
Original & Rehabilitation cost	R 9,762,504

such as loss of production due to ore pass problems and items which are difficult to quantify such as dilution of ore caused by scaling in the ore pass.

If the CAC based lining last three times as long as the OPC option this saving becomes even greater.

6 METHODS OF PLACING ORE PASS LININGS

6.1 Shotcreting

Traditionally the main method of lining an ore pass in the South African mining industry has been by means of dry shotcrete.

In the South African mining industry there has been a marked move towards wet shotcrete over the past few years due to the negative environmental issues associated with dry shotcrete underground, as well as the availability of smaller more economical wet shotcrete units in South Africa.

Most suppliers of pre-bagged concretes to the mining industry in South Africa supply products for wet and dry shotcreting.

6.2 Casting

Casting of an ore pass linings offers the following advantages over placing a lining by means of shotcreting.

Larger aggregate can be used in cast concrete which results in better performance of the concrete when this concrete is subject to abrasion and impact.

Wastage of material is lower due to there being no rebound.

Mixing of the concrete can be better controlled which will result in a higher quality finished product.

When casting an ore pass lining this can be done either by using a sliding shutter or disposable ore pass shutter as illustrated in figure 15.

When the South Deeps Mine lined their new main ore pass system they were faced with the problem that there was not enough shaft time to transport conventional prebagged ore pass lining concretes down the mine. The mine however had a slick line in place which had been used to cast the shaft lining. Because of the superior performance of CAC based ore pass linings achieved in the tests programme the mine had made a decision to use a CAC based concrete ore pass lining. A CAC based ore pass lining concrete had to be developed which could be placed using the mines existing slick line as illustrated in figure 16 above.

The ore pass lining concrete had to be mixed on surface and dropped down the slick line then pumped to the ore pass from the shaft and finally dropped down the ore pass behind the shutters. This involved a distance of around 3000 m. A CAC based concrete



Figure 14. Typical dry shotcreting unit underground.



Figure 15. Disposable ore pass shuttering.

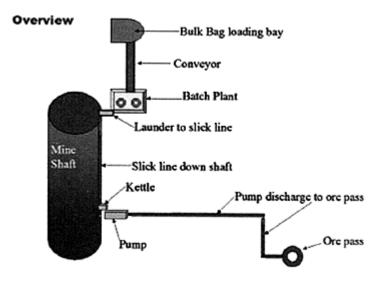


Figure 16. Slick line arrangement at the South Deeps Mine.



Figure 17. Lafarge Aluminates designed slick line ore pass lining concrete.

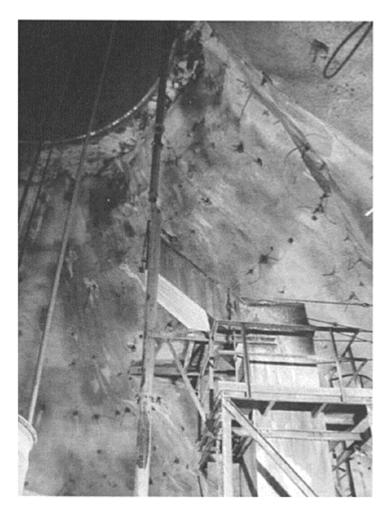


Figure 18. View of slick line transporting the ore pass lining concrete down the shaft.

was developed by Lafarge Aluminates Southern Africa which could be placed using this slick line without any segregation of the mix.

Four placing options are now available to mines when lining an ore pass, these being wet or dry shotcrete, standard casting or slick line casting.

7 HISTORICAL DATA

When deciding on what support type to use to protect an ore pass system it is important to use what historical data is available as the chances are relatively strong that what has worked in the past will again work in the future and what has failed will do so again provided conditions are similar.

One of the best known references of an ore pass which has lasted for the life of the mine is the main ore pass system at Free State Geduld No.5 Shaft which was lined with Corundum Aggregate, Alag® and Cement Fondu® (CAC). (Van Der Westerhuizen 1986). These ore passes were in operation until recently when the mine closed, up until that time over 25 million tons of ore had passed through these ore passes with only one minor repair to a Y-Leg in this ore pass. (Spies 1984).

8 CAC CONCRETE PERFORMANCE

It is widely accepted that calcium aluminate cement based concrete will outperform other equivalent cementitious based concretes in terms of abrasion resistance (Parrish 2002). This has been observed in both phase one and phase two of the testing programme for ore pass lining concretes as discussed in this paper previously.

Exact reasons for the superior performance is not precisely defined but these are a number of theories why this is so.

Scrivener (1999) described how SEM photos show that the higher mobility of the aluminate ions (compared to silica) leads to more widespread deposition of hydrates through the region surrounding an aggregate grain. This may improve the mechanical interlocking between paste and aggregate. This in turn may lead to better adhesion between the CAC and the aggregate. Better adhesion means that aggregate particles are less likely to be ripped away from the concrete and able to give longer service. Figure 19 shows CAC based concrete that has been subjected to an abrasion and impact test. The cement paste binding the aggregate has not been destroyed and no plucking of the aggregate from the concrete can be observed.

If a cementitiously lined ore pass could be kept full of ore and only the same quantity of ore is removed from the ore pass as tipped, damage to the lining will be limited as the ore pass lining will only be subjected to a minor amount of sliding abrasion. Production pressure dictates the amount of ore in an ore pass and the ore pass is usually run empty resulting in the ore pass lining being subjected to severe impact.

One of the requirements for an ore pass lining is high strength concrete usually in the region of 70 to 90 MPa. Some concretes become brittle when designed to reach these strengths which does not present a major problem when subjected to only sliding abrasion but brittle concretes perform poorly when subjected to impact. CAC concretes achieve high strengths without the use of any additives or becoming brittle in nature.

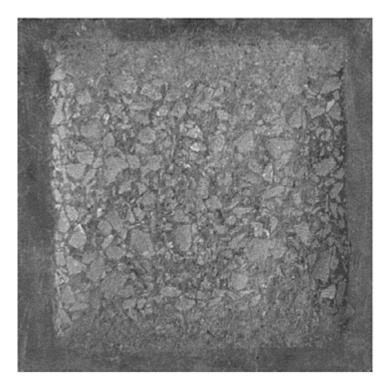


Figure 19. CAC concrete after being subjected to an abrasion and impact test.

9 CONCLUSIONS

In designing an ore pass system it is important that a mine needs to consider not only the type of rock support that must be installed but also the condition of the rock mass and the state of stress.

It is important that laboratory results as well as historical data also be considered in the ore pass design process.

Abrasion caused by both sliding and impact must be considered when designing an ore pass lining.

An abrasion resistant concrete alone is not enough to provide adequate protection to an ore pass. This needs to be combined with quality rock re-enforcement to create a competent system.

The aggregate type is important to the performance of the ore pass lining concrete.

Fibres greatly increase the performance of concretes subjected to abrasion and impact.

CAC based concretes have been proved to have superior performance over equivalent OPC based concretes when subjected to abrasion and impact.

Although CAC concretes are more expensive that OPC based concretes the price differential decreases when the total cost of a project is considered.

The cost effectiveness of a CAC based concrete is realized when the cost of rehabilitation of an ore pass is calculated over its life span.

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Performance assessment of high-tensile steel wire mesh for ground support under seismic conditions

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ABSTRACT: The increasing seismicity experienced by Western Australian underground mining operations in the Yilgarn block was the trigger to investigate the performance of high-tensile steel wire mesh for use as ground support in burst prone mines. The high-tensile mesh Tecco is a chain-link mesh which is made of steel wire with a tensile strength of about 1,800 MPa and includes extra strong border connections. This mesh was especially designed to absorb kinetic energy in rockfall protection systems and showed excellent performance in according field tests. Before the mesh was tested at the dynamic testing facility of the Western Australian School of Mines, preliminary static mesh tests were executed at the mesh testing facility of the Goldfields St Ives Junction Mine. Several different tests were carried out to determine the force-deflection behaviour and the breaking load of the high-tensile mesh under mining conditions, with different bolt patterns and wire diameters. These results are presented in this paper as well as the effect of cut wire to the mesh behaviour and capacity. By using the results of these static mesh tests, it was possible to calibrate a computer simulation which was used to model rockburst impact into the high-tensile mesh. The results of this study show the good performance of the high-tensile mesh Tecco, and the first field trials show that its installation is practicable. High-tensile chain-link mesh can thus be considered as an option for safe ground support in burst prone underground mining areas.

1 INTRODUCTION

The primary ground support in Western Australian underground mining operations mainly consists of weld mesh panels and friction bolts. By going deeper and deeper, the mines in the highly stressed Yilgarn block experience increasing seismic events and according rockburst damage. In order to cope with this hazard, especially designed rock bolts with energy absorption and elongation capacity were introduced. However, in the area of support, the only strategy at the moment is to use thick, fibre or weld mesh reinforced shotcrete which tends to be quite an expensive measure.

Weld mesh panels are easy to handle and are strong and stiff enough to prevent small rockfalls and subsequent unravelling of the drive backs. But they do not have the ability to absorb larger dynamic impacts. The welded connections are brittle and will fail first, followed by the strands with increasing load.

Shotcrete can be applied in different strengths; the strongest being weld mesh reinforced fibrecrete. The big advantage of fibrecrete is that it seals the surface off and prevents unravelling of the rock. But due to its stiffness, the forces acting on it during a seismic impact are getting very high. The results of a dynamic impact are cracks and broken areas with increasing load.

High-tensile steel wire mesh (as the one investigated in this work) showed good performance in rockfall testing. The mesh was able to absorb the kinetic energy by slowing down the impacting rock due to its flexibility. The high strength was necessary to withstand the impact forces and to avoid punching through of the impacting block. These very promising results were the trigger to have a closer look at the application of such mesh for seismic conditions in underground mining.

2 HIGH-TENSILE STEEL WIRE MESH

The Geobrugg Tecco mesh is made of high-tensile wire with a diameter of 3 mm or 4 mm and a tensile strength of 1,770 MPa. Furthermore, this high-tensile wire has an excellent shear and impact resistance. The mesh is square shaped $(100 \times 100 \text{ mm})$ and the wires at the selvedge are bent over and double twisted in such a way that this connection is as strong as the mesh itself (see Figure 1). The mesh is produced on rolls and can be manufactured in widths up to 4 m and in tailormade lengths.

The mesh geometry was designed in such a way that it has a very high breaking load as well as low deflection characteristics to avoid inadmissible deflection rates and unravelling of the rock after a seismic impact.

The static strength of the mesh was determined in several laboratory testing series by Torres (2002) at the University of Cantabria in Santander, Spain. The characteristics for the mesh with 3 mm wires are summarized in Table 1.

The characteristics for the mesh made of 4 mm wire are shown in Table 2.

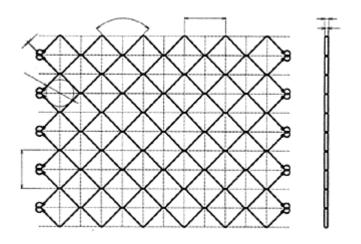


Figure 1. Geometry of TECCO mesh S-95 4 mm.

Table 1. Prope	erties of the	Tecco mesh	S-95 3 mm.
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Clearance	95 mm×95 mm
Wire diameter	3 mm
Wire strength	1,770 MPa
Breaking load of a single wire	12 kN
Tensile strength longitudinal	80 kN/m
Tensile strength transversal	80 kN/m
Weight	1.0 kg/m2

Table 2. Properties of the Tecco mesh S-95 4 mm.

Clearance	95 mm×95 mm
Wire diameter	4 mm
Wire strength	1,770 MPa
Breaking load of a single wire	22 kN
Tensile strength longitudinal	160 kN/m
Tensile strength transversal	160 kN/m
Weight	1.9 kg/m2

Ruegger (1999) tested the mesh in a way quite similar to an application in mining, and stated that the rupture is generally starting at the crossing points but is not sheared over the edge of the plate due to the higher steel quality of the mesh compared to the plate.

3 SETUP OF TEST FACILITY

The herein used test site is located at the Junction Mine of Goldfields St Ives near Kambalda, Western Australia. The test site was designed and used by Thompson and Windsor to test membranes and weld mesh (Thompson et al. 1999).

Figure 2 shows the arrangement for applying load to the mesh. A square loading frame was used to simulate a rigid slab falling into the mesh. A panel of the high-tensile mesh was placed in a convenient size over the steel frame and bolted down to the ground by using four M20 anchor bolts and cone bolt plates $(150 \times 150 \text{ mm})$ in different bolt patterns. The bolt pattern can be chosen either as $1 \text{ m} \times 1 \text{ m}$ or as $1.5 \text{ m} \times 1.5 \text{ m}$ (see Figure 3). A torque wrench was used to apply

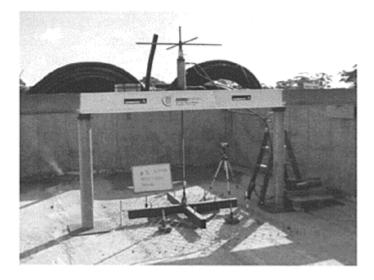


Figure 2. Photograph of mesh testing facility.

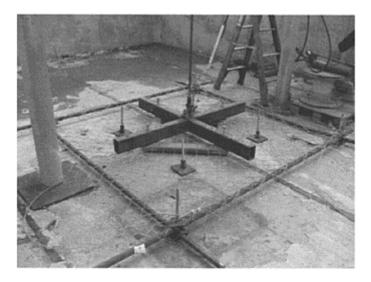


Figure 3. Arrangement of mesh and bolts.

200 Nm of torque to the bolts to provide a pre-tension of about 45 kN.

The loading frame was attached by four bolts to a crosshead which was supported by a Gewi bar. The Gewi bar was attached to a 30 ton hydraulic jack sitting on a strong cross beam above the installed mesh panel. To get initial sag out of the mesh, a manual rotating device was fitted on top of the jack.

An electronic load cell was placed between the nut of the rotating device and the hydraulic jack to measure the applied load. In order to measure the deflection without getting influence of the elongation of the Gewi bar, a tape measure was attached to the bar and the movement was measured by a surveyor's level.

The slip of the mesh relative to the plates was estimated by marking the wires at the start of the test and recording any changes during the test. In order to get a visual impression of the mesh behaviour, a digital camera was placed on a tripod and pictures were taken at regular intervals.

4 PRELIMINARY TESTS

Before starting the actual testing series with a setup as realistic as possible, some preliminary tests were carried out to check the ability of the test site, to calibrate the computer model and to test some specific questions. In order to be able to apply the necessary load to break the high-tensile mesh, several components had to be strengthened up or be improved.

4.1 Calibration tests

The objective of this test series was to determine the force-deflection properties and the breaking load of the mesh and the single wire strands without any border effects in order to calibrate the computer model (see Section 6). For that purpose, the mesh panels were bolted to the ground and only held by the four bolts and plates. The setup can be seen in Figure 4.

Although this is the same setup as the one used in the weld mesh tests of Thompson et al. (1999), the results may not be compared due to the influence of the next row of bolts and plates around a pattern with four bolts. Therefore, it is important only to compare the results from the weld mesh testing with chain-link mesh tests with similar (and realistic) boundary conditions.

Figure 5 shows the mesh during a test without boundary restraint. It is clearly visible that the mesh slides towards the loading point. Although this does not show the behavior of a chain-link mesh installed in an actual tunnel, it gives important input data for the computer simulation. Of course, the deflection values are higher than in reality.

The breaking points were in all cases close to the bolt plate where a force concentration took place. The mesh breaks as Ruegger (1999) stated at the connection points and not at the edge of the plate. Consequently, there is absolutely no need for butterfly plates to protect the mesh from the sharp edges of the plates. Figure 6 shows the effect of the high-tensile mesh on the standard grade steel plates.

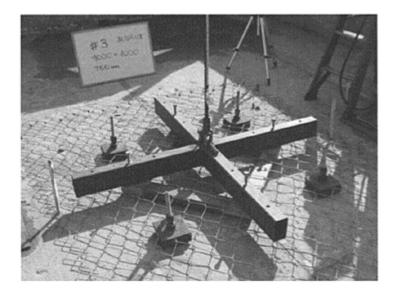


Figure 4. Arrangement of mesh and bolts without any border restraint.

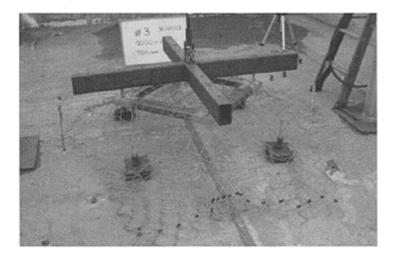


Figure 5. Mesh test with 4 mm hightensile chain-link without border restraint.

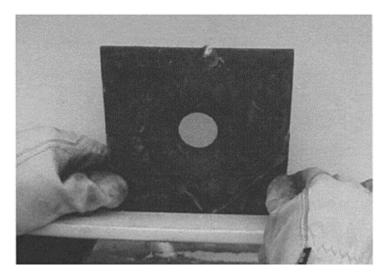


Figure 6. Plate after test with high-tensile mesh.

This series of test further showed the importance of considering the connection of two mesh panels either with enough overlap or with some connection devices. A further test series is therefore planned to test the force-deflection behaviour of different connection types, i.e. more or less meshes overlap or different connection clips (as proposed by Kaiser et al. 1995).

4.2 Tests with cut wire

Ortlepp et al. (1997) stated that chain-link mesh shows the tendency of "unravelling" once a wire failed. In order to test the according behaviour of high-tensile chain-link mesh, a panel was installed and before testing a wire in the very centre of the pattern was cut prior to the test.

Afterwards the panel was tested analogue to the same panel without the cut wire. The results are shown in Figure 8, which shows that there is neither a big difference in stiffness nor in breaking load.

The mesh opens up a bit but then the forces go around the damaged area and the breaking point was again close to the plate totally separated to the area affected by the cut wire. This behaviour confirms the results achieved by Torres (2002b) with the same mesh, with and without cut wire pulled in longitudinal direction.

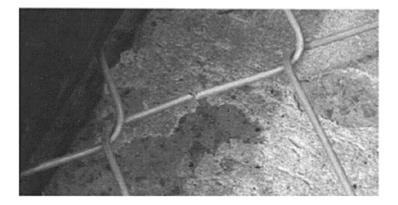


Figure 7. Cut wire in the centre of the mesh panel.

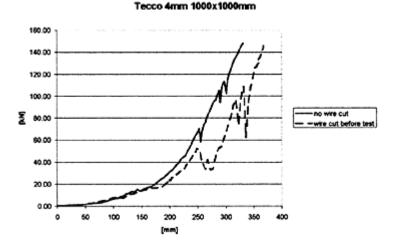


Figure 8. Force deflection graph with/without wire cut.

It can be stated that the phenomena "unravelling" could not be experienced here with the tested hightensile chain-link mesh. These preliminary tests showed that the load can still be increased significantly after a wire breakage until the mesh reaches its final breaking load after several wires broke.

5 MAIN TESTS AND DISCUSSION

As discussed above, the preliminary tests carried out without border restraint did not show the behaviour of mesh that is usually seen underground. In order to be able to get force-deflection characteristics which can be used as a base for dimensioning, the test site was fitted with four border bars. The mesh was now not only attached to the four bolt plates but also to these border bars by using flexible wire rope as shown in Figure 9.

These tests were carried out with the 3 mm and 4 mm high-tensile mesh. Figure 10 shows the 3 mm mesh during a test. The symmetric behaviour and load arrangement is clearly visible.

As experienced with the preliminary tests, the mesh broke at a connection point close to a bolt plate. The breaking load of the 3 mm mesh lies in the order of 70 kN for a 1 m×1 m pattern (see Figure 11) and 80 kN for a 1.5 m×1.5 m pattern with a breaking deflection of 250 mm and 450 mm respectively.

Analogue to the tests with 3 mm mesh, tests with 4 mm mesh were executed as well and showed even stronger performance. The 4 mm mesh reached loads of 160 kN in a 1 m×1 m pattern without the breaking of any wires, with a deflection of about 300 mm (see Figure 12). Although the breaking load could not be reached at the testing facility, it is probable that it is in the range of 180–200 kN.

The average test results are summarized in Figure 13. The force-deflection behaviour of the 3 mm and 4 mm

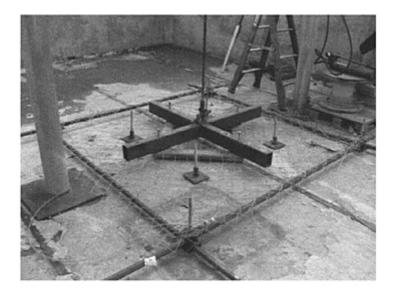


Figure 9. Test setup with border restraint.

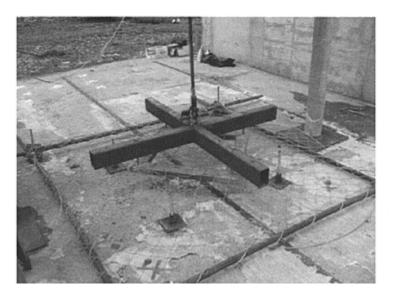


Figure 10. Test with 3 mm mesh and border restraint



Figure 11. Broken wire in the test with 3 mm high-tensile mesh.

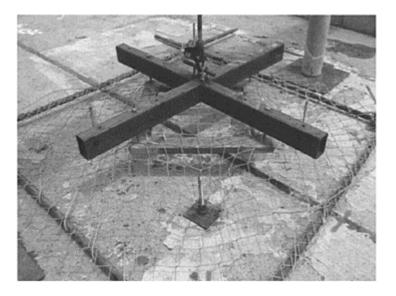


Figure 12. Test with 4 mm mesh and border restraint.

high-tensile mesh is shown for the 1 m×1 m pattern, as well as the results for F51 weld mesh for the same pattern. The F51 weld mesh is commonly used in Australian mines and has a wire diameter of 5 mm

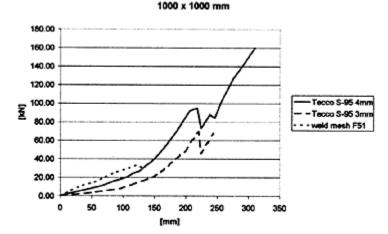


Figure 13. Load-deflectioncharacteristics of Tecco S-95 3 and 4 mm and F51 weld mesh (after Thompson et al. 1999).

with a tensile strength of about 400–500 MPa and wire spacings of 100×100 mm. The results for the weld mesh are taken from the work of Thompson et al. (1999).

The results clearly show the difference between weld and chain-link mesh. The weld mesh has a higher stiffness right from the beginning, but a very low breaking load. The chain-link mesh needs some deflection to activate its strength. The high-tensile chain-link shows quite small deflections (compared to standard grade chain-link mesh) and high breaking loads.

For static applications, the 3 mm high-tensile mesh seems to be the better alternative due to the fact that it is able to withstand a 2–3 m high rock column in a 1 m×1 m pattern. However, with dynamic impacts, much higher forces have to be expected than the static weight of the accelerated rock mass, and for such situations, the 4 mm mesh may be necessary.

6 STATIC SIMULATIONS

For the modelling of the mesh behaviour, the software FARO (falling rocks) from the Swiss Federal Institute of Technology ETH in Zurich was used (Volkwein 2003). It was developed to simulate rockfall impacts into flexible rockfall barriers and is based on rope elements and a discrete finite element method (time stepping). From time step to time step, the material laws of the single elements are used to calculate the reaction in these elements.

By modelling the test setup as used in these quasi-static mesh tests and introducing a constant speed to the test frame, it is possible to simulate the tests described in Section 5.

To calibrate the material properties of the mesh elements, the results of the preliminary tests were used. Figure 14 shows the modelled test setup.

Both the load deflection characteristics of the single mesh elements have to be determined as well as the breaking load of these elements. The first simulations

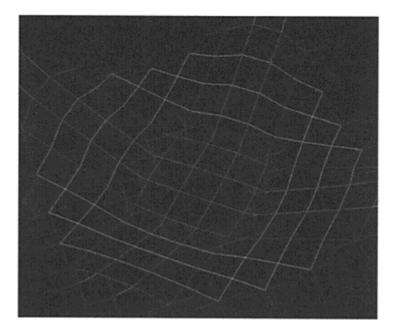


Figure 14. Test setup modelled with FARO.

show that the executed tests can be simulated quite well and that the deviation of the computed results is in the range of 10% compared to the test results.

With these simulations it is possible to calculate the deflection of any bolt pattern by introducing a certain load. Consequently, it would be possible to design a bolt pattern with an expected input load and a permissible deflection. Of course, also the according load on the bolts can be calculated.

7 DYNAMIC SIMULATIONS AND TESTS

Since the FARO software was developed to simulate dynamic impacts by using material laws gained from quasi-static component tests, it is possible to simulate dynamic impacts into the mesh system. The first simulations showed energy absorption capacities of the 4 mm mesh of up to 50 kJ in a 1 m×1 m bolt pattern.

But in order to verify these results, dynamic mesh tests are necessary. For that reason, tests with hightensile chain-link mesh are planned at the Western Australian School of Mines, once their test site is equipped for dynamic support tests. Furthermore, it will be

possible to compare the different support types as shotcrete, fibrecrete, membranes, weld and chain-link-mesh, under similar dynamic conditions.

8 CONCLUSIONS

The test work presented clearly shows the performance of high-tensile chain-link mesh for underground applications where high loads have to be expected. Compared to weld mesh, high-tensile chain-link has no welded or other weak points. In addition, it has a very high shear and impact resistance and strong end connections.

The load-deflection characteristics are promising that the breaking loads are well above the required values and also the deflection is in the range of permissible values. By using the according simulation software, a way of dimensioning bolt patterns and energy absorption capacities was presented.

The first installation trials showed that it was practicable to install the high-tensile mesh which is made of rolls and not panels. However, since most of the Western Australian mines are equipped and trained to install rigid weld mesh panels, some more work has to be done on the installation process.

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12 Other support

Pillar replacement using pre-stressed timber props

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ABSTRACT: Significantly increased ore extraction is possible in room and pillar mining if ore pillars can be reduced. Pre-stressed timber props were successfully used to provide stope hangingwall support in a shallow dipping portion of the narrow North-Eastern orebody at the Rav 8 nickel mine in southern Western Australia. Extraction ratios exceeding 90% were achieved using the system (compared to an expected ratio of approximately 80% using a conventional pillar configuration). An additional benefit was gained due to a reduction in mining dilution.

1 INTRODUCTION

A cost and time efficient artificial support system was required for attempts to maximise ore recovery and reduce mining dilution from shallow dipping nickel lodes at Tectonic Resources NL's Rav 8 Nickel Project near Ravensthorpe, Western Australia. Locally available timber props (*pinus radiata*) were used in conjunction with a proprietary waterbased hydraulic pre-stressing unit.

2 BACKGROUND

2.1 Geology

The project area is comprised of a classic Archaean granite-greenstone terrain. The deposit occurs within an extensive ultramafic flow unit which is a member of the Bandalup Ultramafics. These ultramafic rocks are generally inter-layered with tholeitic metabasalts and felsic volcanic rocks. The ultramafics are interpreted as being one major komatiite flow sequence which repeats in outcrops as a result of thrust stacking.

There are primarily two massive nickel sulphide lenses being exploited at the Rav 8 mine (Figure 1). These lodes occur in an east-west shear zone that dips flatly to the south. The main lode occurs on the interface between the ultramafic hanging wall and the felsic footwall rocks. Both lodes plunge to the south-east. The second lode, locally known as the NEOB (North-East

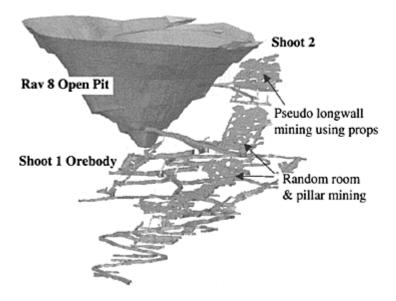


Figure 1. Rav 8 Mine layout (perspective).

Orebody), has been structurally emplaced under a felsic volcanic hangingwall with the same felsic rocks in the footwall. Late stage normal faulting commonly displaces and disrupts the ore surface.

In the upper levels of this ore-shoot the tenor of the ore is below average. An average pay run would return lode widths of 0.40 m and nickel grades between 10–12%. The dominant sulphide assemblage is violarite-pyrite, changing to pyrrhotite-pentlandite in the lower levels of the mine. The hangingwall of the upper NEOB is formed by a sharp, but sometimes stepped, contact with a felsic unit. A lode-parallel, south-east dipping foliation fabric, and sub-vertical to very steeply north-west dipping faults are the other dominant geological structures in the lode hangingwall.

2.2 Mining

Originally ore strike development was performed either by single boom jumbo or hand held machines. All hand held stoping was carried out utilising a random room and pillar method.

The pre-stressed timber props have been most effectively used within the NEOB where the key to stope stability was to maintain the integrity of the felsic hangingwall by

providing regular support. Particular care in providing support around steps in the hangingwall contact was essential.

The upper NEOB was mined by rising up-dip from the main sub-level drive to the overlying sub-level. The number of rises mined depended on the local strike width of the ore zone. Intermediate sub-level drives were developed to join the rises. This configuration extended to the pinch-points of the mineralisation. Rooms were formed by stripping the panels between the strike drives and rises. Broken ore was mucked using scrapers. Hangingwall support was provided initially by *in situ* pillars.

This method commonly resulted in an average stope height of around 1.8 m and a primary extraction rate no better than 75%.

In the secondary phase of stoping in a given area, artificial support in the form of prestressed timber props was introduced, and pillars were progressively mined, either partially or totally. With the introduction into the mine of pre-stressed timber props, secondary extraction within these random room pillar stopes was readily achieved, significantly improving the total extraction rate to better than 85–90%.

In later mining areas a pseudo-longwall mining method was attempted. A slot rise was pushed up dip from the level. At every twenty metres up-dip a sublevel was created. This formed up the longwall face that was then pushed along strike perpendicular to the strike drive. Horizontal holes were bored out of the slot and fired directly toward the drive. As the face advanced a line of pre-stressed timber props was placed in the stope on a 2 m×2 m spacing. Broken ore was mucked using scrapers and high pressure water sluicing. Typically the props negate the need to rock bolt within the stope. As a result, the mining height was able to be reduced to the lode width or a minimum of 1.2 m.

Using this technique the primary extraction improved to be better than 95%. Rock pillars were left in place only in significant low grade areas or where there was a need to stabilise a particularly large brow that may have formed in the hangingwall.

The reduced mining heights meant that stoping could be pushed further into the marginal ore on the periphery of the main ore shoot of the lode, again leading to a higher overall extraction of the resource.

The reduced mining height also provided a significant reduction in dilution and an improvement in control over dilution.

2.3 Stope stability conditions

Stope hanging walls and the backs of the on-dip decline were mined with the aim of breaking to persistent "shear" structures located at or near the ultramaficfelsic contact. The hanging walls thus formed were generally sound.

However, there were areas and conditions which were recognised as having potential to adversely influence hangingwall stability; namely where:

- 1. Two or more hanging wall structures existed, promoting slabbing following exposure and relaxation with time.
- 2. Sub-vertical faults defined potentially unstable slabs.
- 3. Major structures oblique to the hangingwall (splays) were encountered locally.
- 4. Stoping approached the pinch-lines of the lode where ground conditions were typically poorer.
- 5. Time-related deterioration occurred.

Potential for pillar instability was also recognised. The hangingwall and footwall contacts with the lode were typically sheared and comprised approximately 0.1 m thick zones of poor quality materials; however, these zones rarely showed signs of displacement or disturbance. Damage to pillars was more likely to result from blast effects and/or over-excavation.

3 INTRODUCTION OF PROPS

The observed lack of stress-induced effects on *in situ* pillars in the upper north-eastern lode suggested that pillar replacement would be a viable means of increasing ore recovery.

Hangingwall stability was able to be maintained around damaged pillars, usually with only minor increases in installed rockbolt reinforcement (typically 1.8 m long resin bolts installed at $\leq 70^{\circ}$ to the hangingwall).

The few stope hanging wall failures that occurred were shallow (≤ 1 m thick), localised (a few square metres in area) and directly related to major geological structures.

On the basis of these observations it was inferred that a relatively shallow arch formed in the competent felsic hangingwall rocks across the stoped out panel and that props could be used to replace pillars provided every practical effort was made to ensure the integrity of the stope hangingwall.

The props needed to be capable of carrying the dead load of a few metres depth of rock (into the hangingwall) and would assist in maintaining hangingwall integrity.

3.1 Selection of props

Pine (*pinus radiata*) props were readily available from a plantation approximately 180 kilometres from the mine. Green, untreated logs of \geq 150 mm diameter were selected for use. This timber was expected to yield progressively and visibly under a gradually increasing load. Given the relatively short stope lives, no advantage would be gained by using treated timber.

The grain-parallel compressive strength of the logs was assessed in the laboratory by loading a series of approximately 150 mm diameter, 370 mm long samples to rupture. The mean ultimate load was approximately 700 kN (and mean compressive stress approximately 38 MPa). Young's Modulus was assessed at approximately 10 GPa. The critical prop lengths for buckling are thus (approximately) 1.9 m, 2.2 m and 2.5 m for 150 mm, 180 mm and 200 mm diameter props respectively.

Actual prop diameters varied from 150 mm to approximately 190 mm. The larger diameter props were used in the thicker portions of the North-East Lode.

3.2 Prop pre-stressing system

A prop cap pre-stressing system was preferred over the older wedge prop setting method in that it provided immediate active hangingwall support, easier setting, more reliable and uniform contacts between the prop and the rock, and initial uniform axial prop loads. A proprietary water-based hydraulic prop cap system, the Jackpot® Model 190 was used. The prop cap is shown schematically in Figure 2. A pre-load of approximately 20 tonnes (~200kN) was applied to each prop.

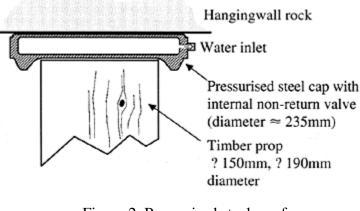


Figure 2. Pressurised steel cap for timber prop.

4 USE OF PRE-STRESSED PROPS

It is preferable to plan a systematic (and conservative) array of *in situ* pillars and to adhere to the development of that array. Subsequent pillar reduction or removal should also be systematic.

The props were used throughout the rooms in which pillars were to be reduced or removed. This general approach permitted wider pillar spacings even in areas where pillar mining would not be performed. A nominal prop pattern of 2.5 m×2.8 m was used. The ultimate capacity of this pattern of support was theoretically that of the dead load of the immediate (approximately) 3.5 m of rock above the hangingwall (that is, the system could support rock to approximately 2.3 metres above the hangingwall at a Factor of Safety of 1.5).

Where pillars were to be recovered or partially recovered, props were placed prior to stripping.

The hanging wall was re-scaled to ensure that there were no loose slabs present. The floor was cleared to ensure that the prop has direct, clean contact with the stope footwall.

Individual props were cut to length underground (on the job) after measurement of the hanging wall-footwall separation. With the Jackpot fitted, the prop is hammered into place (perpendicular to the hanging wall), the water pump attached and the Jackpot is pressurised. Hand-pumping was continued until a pressure relief valve opened at a pre-set pressure/load (20 tonne pre-load in this case). This simple method ensured that the required pre-load was achieved for all props. The Jackpot could be placed against either the hanging wall (as was the case at Rav 8) or the footwall. The system used Jackpot 190

pre-stressing caps which were placed directly against the hangingwall where formed by felsic rocks (Figure 3). To avoid potential punching shear failure, headboards were used

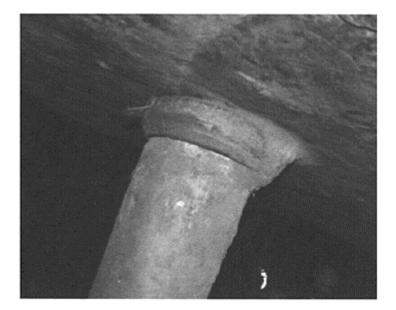


Figure 3. Jackpot® prop cap in position (this prop was immediately adjacent to a pillar firing).

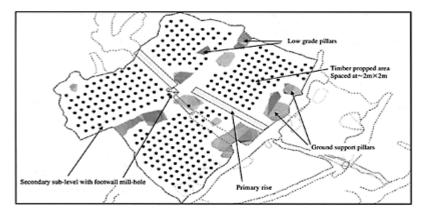


Figure 4. Stope extraction using props (89.2% extraction, increased to 92.6% if low grade pillars mined).

to spread the load where the hangingwall comprised or contained weaker ultramafic rocks.

Particular care was taken in setting the props on a stable, even footwall to avoid eccentric loading and potential prop "slip".

It was also important when installing new props, that the Jackpots on props immediately adjacent to new props were re-pressurised, since it was possible that the newly installed support could displace the hangingwall and reduce the load on the previously installed props.

5 STOPE STABILITY PERFORMANCE

The typical dimensions of panels from which pillars were removed were 40 m along strike and 35 metres up dip. The maximum area from which all pillars were removed was 50 m along strike by 18 m up dip.

In areas where a long-wall mining method was applied from commencement, extraction rates better than 90% were commonly achieved, with better than 95% attained at times. When returning to areas which were primarily mined as random room and pillar stopes, secondary extraction increased total extraction to better than 90% (excluding pillars left as subgrade). An example of the extensive extraction able to be achieved is shown in Figure 4. In this case, an extraction of 89.2% was achieved, and could have been increased to 92.6% if low grade pillars were mined.

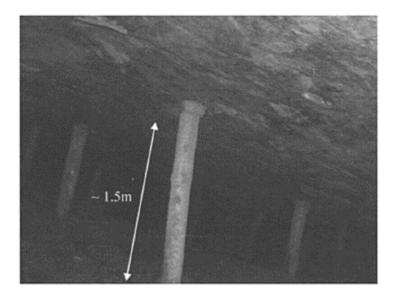


Figure 5. Installed props with Jackpot® caps.

A typical propped stope is shown in Figure 5 and yielding props are shown in Figure 6.

6 GENERAL COMMENTS

The critical factor for stability was to maintain the integrity of the competent felsic hangingwall. In this respect, it was crucial that the hangingwall was mapped regularly and potentially unfavourable geological structures were detected so that appropriate measures could be implemented; for example, an



Figure 6. Props yielding.

increase in the number of props used and/or a decrease in pillar scavenging.

Establishment of a regular prop grid was important and frequent inspection of the conditions of props in active stoping areas was essential. Once planned pillar recovery in an area was completed, the area was abandoned and cordoned off. Pillar scavenging did not exceed the pre-planned limit.

A small number of areas of localised hangingwall instability were detected during stoping. These zones were typically identified as problematic prior to failure and were controlled by re-propping and/or by the installation of additional props and rockbolt reinforcement. However, the level of reinforcement able to be practically installed in the stopes was adequate only for highly localised remediation.

Onset of prop failure was detected in some areas. The prop failure mechanisms were compressive rupture, bending due to eccentric loading and some buckling. Procedures required that an area would be abandoned if $\geq 20\%$ of props showed distress (cracking due to compression or bending). Similarly, an area would be abandoned if replacement or additional props showed signs of being over-stressed.

A major hangingwall collapse occurred as stoping neared completion in the upper NEOB. The extent of the collapse was limited by the competent rib and sill pillars separating the stoping panels. It was considered possible that the hangingwall collapse could induce subsidence of the overlying rocks (due to the expected inability to form a competent arch in the overlying near-surface rocks); however, to date such a collapse has not occurred.

The major lesson learned from this event for future room and pillar mining incorporating pillar recovery using props was that the final stage of hangingwall failure in an extensively mined area could be sudden, rather than progressive, and could involve a large area. There are thus stringent requirements for reentry procedures into stope areas. Mining strategies need to be defined prior to stoping and (pragmatically) adhered to during mining. Mining must retreat away from the large spans formed during stoping. Re-entry into extensively mined areas to undertake additional pillar recovery/trimming and/or stope clean up is considered to be potentially hazardous. There must be a finite cut off limit, related to the percentage of the stope extracted, for allowing re-entry into stope blocks.

With the longwall mining approach, experience has shown that in areas with poor hangingwall conditions use of an increased level of pre-stressed prop support can provide a better result than resort to an *in situ* pillar or pillars. Pillar development slowed production and the required extra drag blasting exacerbated hangingwall stability conditions.

7 CONCLUSIONS

Significantly increased ore extraction is possible in shallow dipping room and pillar stoping using prestressed timber props to provide hangingwall support. Extraction ratios exceeding 90% were achieved using the system. Additional benefit can be gained via reduction in mining dilution.

However, monitoring is required to help understand and/or predict stope hangingwall behaviour. Accordingly an initially conservative approach to retreat and/or re-entry is advised. It is suggested that an initial extraction percentage be nominated at which point stability conditions are reviewed and future stoping strategies are decided.

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A probabilistic approach to determining stable inter-pillar spans on Tau Lekoa Mine

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AngloGold Ashanti Limited, South Africa

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ABSTRACT: Since the commencement of mining at Tau Lekoa Mine in 1991, large wedge or dome collapses have been problematic. Crush pillars are used to prevent these and the current layout has been derived through iterative design. The span between crush pillars is based on an empirical relationship between the fallout thickness and span. Crush pillar spans are designed to limit the potential fallout height to a thickness that is controllable by means of internal support. The correlation between the span and fallout height is statistically poor and may mask underlying contributing factors. Observed thickness often deviate significantly from the design prediction and seem to cluster in certain areas. Geotechnical and mining data has been collected and statistical distributions have been determined to allow a probabilistic approach to the design of stable spans. A statistical analysis of eighty large collapses has also been conducted. Use of programs such as @RISK and J-Block have been made in assessing the risk associated with different spans and geotechnical conditions. Ultimately, the aim is to develop an optimised pillar layout that is beneficial from both a safety and economic perspective.

1 INTRODUCTION

1.1 Location and description

Tau Lekoa Gold Mine (AngloGold Ashanti Limited) is situated approximately 170 km southwest of Johannesburg, near Orkney in the North West Province of South Africa. The mine has been in production since 1991. A scattered mining strategy using predeveloped access tunnels is employed on the mine, due to the geological complexity of the orebody. Stoping operations take place on strike with raise lines spaced 180 m apart on strike. Mining is conducted at depths between 900 m and 1650 m below surface (Dunn, 2004).

1.2 Geology and geotechnical environment

The Ventersdorp Contact Reef (VCR) is mined at Tau Lekoa Mine. The VCR lies unconformably on the Gold Estate Formation of the Central Rand Group and is overlain by the Klipriviersberg Group of the Ventersdorp Supergroup.

The orebody can be described as tabular, dipping at $\pm 30^{\circ}$ towards the northwest. The channel width varies between 10 cm and 300 cm and reef rolls are common. Several major faults striking mainly north-east to southwest and dipping southeast disturb the area. Quartz and calcite veins, dykes and joint sets also intersect the reef. The veins are often flat dipping and are especially hazardous. Two major joint sets have been identified, one striking north to south and the other northwest to southeast. Both sets are steep dipping at 70–90°. Low angle thrust faulting has also been observed. In some areas a mylonite filling is present between the lava and the VCR. The mining lease can be sub divided into four depositional settings:

- An upper terrace
- Middle terrace and slope areas
- Main channel
- Reworked channel areas

Stress measurements conducted on 1200 level in 1988 (Lombard, 1989) indicated the following:

- A relatively normal near vertical stress of 30-40 MPa.
- Relatively high horizontal stress of similar value, acting approximately north to south.
- Low horizontal stress acting approximately east to west. Low angle stress fractures observed parallel to stope faces and scaling in the rock passes are indicative of a high horizontal stress in the north south direction.

At various stages, testing of core has been conducted to determine the uniaxial compressive strength (σ_c) of the lava hangingwall and footwall. According to Lombard (1989), the lava σ_c varied between 180–280 MPa and the immediate footwall varied between 150–220 MPa. Tests conducted later, indicated average σ_c values of 180 MPa, 155 MPa and 125 MPa for VCR, hangingwall lava and footwall quartzite respectively (Rosenblatt, 1994). Point load tests conducted on hangingwall lava indicated an average σ_c of 240 MPa. Several lava types were identified and the σ_c values varied between 185–330MPa (Fourie, 1999).

1.3 Background of ground control problems

Large falls of ground (FOG) have occurred since the commencement of mining at Tau Lekoa Mine. Since 1991, 108 large FOG of ground have been observed. Generally, any collapse exceeding 10 m^2 is considered large. Fairly reliable and complete data is available for 88 of these incidents. Generally, these falls have been geologically controlled wedges or dome structures that are difficult to identify or predict.

When mining commenced in 1991, the initial method was down dip mining, leaving crush pillars 30 m apart with timber composite packs as in-stope support. The first collapses occurred when the faces had advanced ± 30 m. In-stope pillars were then

introduced at irregular intervals but collapses continued to occur when certain spans were exceeded.

Due to poor efficiencies associated with cutting pillars when mining down dip, the mining method was changed to breast mining with crush pillars left on strike. The maximum stable span was determined to be 20 m through back analysis (Harris & Rosenblatt, 1993). The in-stope support was changed to elongates at mining heights less than 1.8 m, and 3 m long resin bolts for mining heights exceeding 1.8 m after the pack support was found to be ineffective due to low closure rates.

Collapses continued to occur and mid-panel pillars were left at irregular intervals. Mid-panel pillars became a standard in 1995. In-stope support was changed from normal to yielding elongates, and 1.5 m long rockbolts with cement based grout (full column) replaced the 3 m resin bolts. Pre-stressing devices (PSU) were introduced onto the yielding elongates at the end of 1997 in an effort to reduce blast outs and increase the initial stiffness.

2 STABLE INTER-PILLAR SPANS

Stope panel spans can be defined as the distance between support pillars within or delineating a stope panel. The stability of stope panel spans is an important aspect of any pillar supported mining system. At Tau Lekoa Mine the term inter-pillar stability is often used when referring to stable stope panel spans.

The stability of the span between pillars is a function of the pillar spacing and the ability of the internal support to control or prevent large-scale panel collapses (Haile & Jager, 1995). The design of safe panel spans must take into account:

- Variability in the immediate beam thickness
- Intensity, orientation and alteration of hangingwall jointing
- Rock strength
- Horizontal stresses
- Key block failure and block dimensions

2.1 Failure modes

Haile & Jager (1995) identified six different modes of failure in pillar supported hard rock mines within the Bushveld Igneous Complex (BIC).

2.1.1 Key block failure

Where two or more mutually intersecting joints are present in the stope hangingwall creating an unstable block geometry.

2.1.2 Wedge failure

Where two major planes of weakness intersect in the stope hangingwall. The areal extent of the failure is generally far greater than that of key block failure.

2.1.3 Buckling failure

When the hangingwall beam buckles and failure is not defined solely by joint geometry.

2.1.4 Beam failure shear failure

Failure occurs due to slip on widely spaced and subvertical planes of weakness or initiated as fractures close to pillars or abutments.

2.1.5 Cooling dome failure

Failure is initiated due to fallout on shallow dipping joints on the periphery of an upsidedown basin shaped block of rock. Domes are approximately circular in shape and vary in size from a few square metres to several hundred square metres and are common across the whole of the BIC.

2.1.6 Unravelling failure

Occurs when the hangingwall of the stope contains a prominent joint set of uniform dip and direction and the hangingwall span between pillars exceeds a certain critical limit.

2.2 Failures at Tau Lekoa Mine

With the possible exception of buckling failure, all of the above failure modes have been observed on Tau Lekoa Mine. In terms of dome failures, the upper limit size limit appears to be tens of square metres as apposed to several hundred square metres. The dome structures are related to both cooling and flat faulting along the VCR lava contact.

In some cases, more than one failure mode has been observed. For the purposes of this study, the focus will be on the larger types of failure, namely:

- Wedge failure
- Dome failure
- Beam failure
- Wedge/dome failure

2.3 Design of stable inter-pillar spans

Several different approaches can be used to design stable inter-pillar spans and these are outlined below.

2.3.1 Rock mass classification systems

The use of rock mass classification systems appears to be wide spread and has been used in a wide range of mining environments to determine stable spans. Swart et al (2000) reviewed the following four systems for evaluating the stability of panel spans:

• Geomechanics Classification or Rock Mass Rating (RMR) system (Bieniawski, 1973).

- Norwegian Geotechnical Institute (NGI), rock quality index or Q-system (Barton et al, 1974).
- Mining Rock Mass Classification or Modified Rock Mass Rating (MRMR) system (Laubscher & Taylor, 1976).
- Modified Stability Graph Method using the Modified Stability Number (Mathews et al, 1981).

Over the years some of these systems have been modified to suit local conditions. Watson & Noble (1997) reviewed several systems and concluded that the Modified Q-system provided the most accurate description of observed conditions. However, it did not take into account stress influences and discontinuity orientation.

York et al. (1998) reviewed various rock mass rating systems available and found the Impala Platinum Mine adaptation of the Q-system resulted in the best correlation between the rating and actual observed conditions. This system had been adapted to take into account major unfavourable geological structures (Human, 1997) and was applicable to platinum mines mining the Merensky Reef (MR).

A Critical Panel Span Design Chart based on an analysis of stable and collapsed panels on the MR was developed. From this work it was concluded that some collapses did not agree with the chart and that a greater understanding of the rock mass was required specifically in terms of stress conditions and discontinuity persistence and orientation.

From research on various rock mass classification systems, several conclusions were reached and are covered in detail by Swart et al (2000). However, in terms of the Tau Lekoa problem, the most poignant was that rock mass classification systems only describe rock mass failure and do not consider other potential failure mechanisms such as beam, block, wedge or dome failures.

2.3.2 Analytical methods

This includes methods such as numerical modelling, keyblock analysis and beam analysis. Elastic beam analysis theory is useful in explaining roof failure of bedded deposits. Where sub-vertical jointing is present the 'voussoir' beam theory can be applied (Swart et al., 2000, Beer & Meek, 1982).

The application of keyblock methods is dependent on the correct interpretation of the structural geology and the identification of unstable wedges and blocks. Conventional deterministic key block analysis as described by Goodman & Shi (1985), the natural scatter is often ignored and mean values used. Joint continuity is also disregarded. Probabilistic methods, such as J-Block (Esterhurizen, 1996) can be applied to overcome these problems.

2.3.3 Statistical or experience based design

A survey conducted by Haile & Jager (1995) indicated that the design of stope panel spans was primarily based on experience gained in a particular environment over the years. In many cases, the use of unstable versus stable spans statistics form the basis of stope panels span design. Harris & Rosenblatt (1993) applied this approach on Tau Lekoa

Mine. This paper focuses on this approach and attempts to set the groundwork for a probabilistic approach to the design of stable spans.

3 TAU LEKOA STOPE SUPPORT STRATEGY

3.1 *Pillar layout*

The Tau Lekoa Mine Stope Support Strategy incorporates regional and local support (Dunn, 2003). A conventional strike crush pillar layout is used with a face length of 20 m with mid-panel pillars every 10 m or 16 m depending on the mining direction and stope width (Figure 1).

Regional support is provided by 10 m wide squat pillars (width to height ratio of >5) left every second panel on dip and 120 m apart on strike. At less than 1200 m below surface, these pillars are positioned 3.5 m from the raise. At depths, exceeding 1200 m these pillars are positioned 10 m away from the raise to prevent damage to the raise (Leach, 1998).

The crush pillars where designed to have a factor of safety less than one to ensure crushing and have a width to height ratio of 1.5 to 2. The peak and residual pillar strengths are estimated to be 80 and 10 MPa respectively (Rosenblatt, 1994).

3.2 Pillar spans

Inter-pillar spacing has been derived from back analysis of large collapses based on a relationship between the fallout height (thickness) and span (Dunn, 2000; Dunn 2003). By plotting these two parameters against each other, an empirical relationship has been determined (Figure 2). The fallout height was initially estimated to be 0.125 times the span. Additional work indicated that the fallout height be 0.11 times the span (Rosenblatt, 1994). Recent work by the author on a modified database indicates a similar value. For support design purposes, the original factor is used as this provides an additional factor of safety.

Further analysis of large collapses was conducted by plotting their occurrence on a plan. It was observed that approximately half the large collapses occurred at stoping widths greater than 1.8 m, and that the occurrence of large collapses in south mining panels was double that in north mining panels when the data was normalised relative to production (Judeel & Laas, 1999).

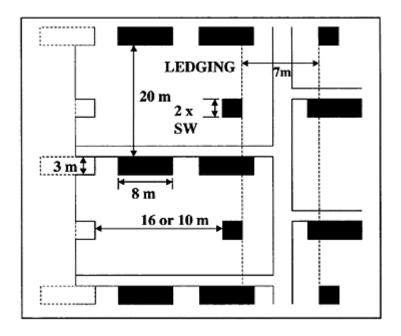


Figure 1. Crush pillar layout.

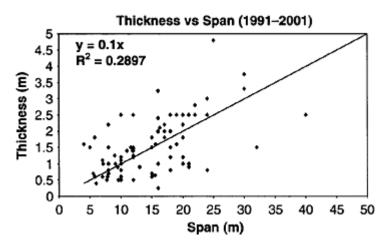


Figure 2. Fallout height versus span for large collapses (1991–2001).

3.3 In-stope support design

Three in-stope layouts and standards have been developed to cater for conditions at Tau Lekoa Mine. These standards take into account fall of ground heights for all stoping fatalities since 1991 and predicted height of large collapses.

3.3.1 North mining at stoping widths less than 1.8 m

Maximum face length of 20 m with mid-panel pillars spaced a maximum of 16 m apart on strike. In-stope support consists of pre-stressed elongates spaced 1.5 and 2 m apart on dip and strike respectively. This system is sufficient to support a thickness of at least 2.4 m.

3.2.3 North mining at stoping widths exceeding 1.8 m

Maximum face length of 20 m with mid-panel pillars spaced a maximum of 10 m apart reducing the potential fallout to 1.25 m. Grouted rockbolts, 1.5 m in length, spaced 1.5 and 2 m apart on dip and strike respectively are used.

3.3.3 South mining at stoping widths less than 1.8 m

Generally, poorer ground conditions are encountered when mining south due to unfavourable joint directions relative to the stress field. This problem is exacerbated when mining towards the north occurs first. Where possible, south mining is avoided. When south mining is unavoidable, the mid-panel pillars are reduced to 10 m thus reducing the maximum potential fallout to 1.25 m. Elongates are spaced 1.5 m apart on dip and strike and the stoping width is limited to 1.8 m.

3.4 Fallout height versus span relationship

This relation has a poor correlation with a coefficient of determination (r^2) of 0.29 (29%). An analysis was conducted to determine if this relationship varied for different directions and if the introduction of midpanel pillars in 1995 had any influence. The results are summarised in Table 1. The relationship does not

Description	Relationship	r^2
All mining: 1991–2001	H=0.1×S	0.2897
All mining: 1995–2001	H=0.0977×S	0.174
North mining: 1991–2001	H=0.1047×S	0.4052
North mining: 1995–2001	H—0.0988×S	0.1935
South mining: 1991–2001	H=0.0871×S	0.2981
South mining: 1995–2001	H=0.0917×S	0.4888

Table 1. Summary of fallout height (H) versus span (S).

Centre gullies: 1991–2001	H=0.108×S	0.086
Centre gullies: 1995–2001	H=0.1035×S	0.4976

vary markedly although the correlation ranges from 8.6% to 50%.

4 STATISTICAL ANALYSIS OF LARGE FOG

A statistical analysis has been conducted on the dimensions of these large collapses. This analysis considered the period from 1991–2001 and the period 1995–2001 to determine if the adoption of mid-panel pillars had an influence. The analysis was conducted for panels mining in a northerly and southerly direction, centre gullies (CG) and for all panels combined.

Histograms indicating the frequency and cumulative distributions were compiled in Excel, using @RISK to fit distributions to the data. As the database does not indicate the orientation of the lengths and widths, it was decided to combine the lengths and widths and this is called the fall of ground (FOG) dimension, which gives an indication of the range of dimensions.

4.1 Fallout Height

Table 2 summarises the statistical data for fallout height for the different categories of large collapses (Dunn, 2004). Figure 3 shows the cumulative fallout height for all collapses and Figure 4 indicates the distribution fit from @RISK. This data can be used to determine support requirements in terms of support resistance requirements and in the case of tendons the required length.

The mean fallout height for centre gullies is higher than for the other categories, which is unexpected as the centre gully span specified in the standards is smaller than required in normal panels. The largest fallout height was recorded across a centre gully. This can possibly be related to inadequate tendon length. The standards require 2.2 m long tendons in

Period	Mean	Std Dev.	Max.	Min.	Fit
All:	1.62	0.92	4.8	0.25	Inverse
1991–2001					Gaussian
All:	1.45	0.73	3.25	0.25	Beta
1995–2001					General
North:	1.54	0.97	3.75	0.25	Beta
1991–2001					General
North:	1.37	0.94	3.25	0.25	Inverse

Table 2. Summary of fallout height statistical data.

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1995–2001					Gaussian
South:	1.49	0.72	3	0.5	Inverse
1991–2001					Gaussian
South:	1.5	1.62	3	0.5	Log
1995–2001					Logistic
CG:	1.93	0.85	4.8	0.6	Extreme
1991–2001					Value
CG:	1.71	0.54	2.5	1	Extreme
1995–2001					Value

the centre gully hanging wall with elongate support on the ledge shoulders.

The distribution of the deviation of actual thickness from the predicted thickness is shown in Figure 5. This is an indication of the probability of a large collapse exceeding the expected thickness. For north mining at less than 1.8 m, there is a 25% chance of exceeding the predicted fallout height of 2 m and a 12% chance of exceeding the 2.4 m fallout height the support is capable of controlling.

For mining at greater than 1.8 m stoping width there is an 18% and 12% chance of exceeding the predicted thickness and the rockbolt length respectively. These percentages are relatively high but as large

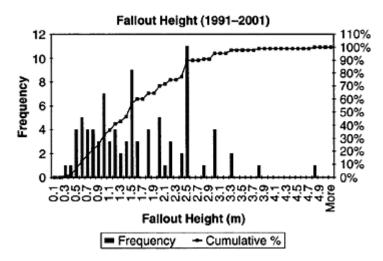
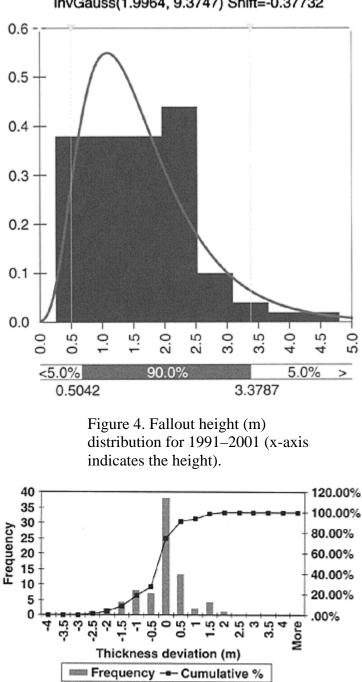


Figure 3. Fallout height for all large collapses (1991–2001).



InvGauss(1.9964, 9.3747) Shift=-0.37732

Figure 5. Deviation of actual fallout height from the predicted fallout height.

collapses are a small percentage of all FOG, the 95% criterion is probably satisfied (Esterhuizen, 1999).

In 66% of cases, the fallout height was less than the predicted thickness. This indicates that either the instope support standards are inadequate or that support standards are not applied properly. The latter is probably the cause as the standards have been designed with a considerable factor of safety. This aspect needs to be investigated further.

Another approach to determine the fallout thickness height that should be used for support design purposes, is to determine the factor for each large collapse. The mean, standard deviation and 95% confidence level could then be determined. For all large collapses, the factor is 0.218 at a 95% confidence level. At a 16 m span, the support would have to cater for a fallout height of 3.49 m. This possibly explains why some of the collapses occurred.

4.2 Spans

Table 3 summarises the statistical data for panel spans associated with large collapses. Excessive spans seem to be a major contributor to large collapses. For collapses in north panels, south panels and centre gullies, spans exceeded the standard in 39%, 79% and 86% of cases respectively.

In total 63% of large collapses spans exceeded the standard. Figures 6–8 show the distribution of actual pillar spans measured from plan, for a 20-month period (March 2000 to November 2001). These distributions indicate that 48% of south panels, 67% of north panels with a stope width exceeding 1.8 m, and 8% of north panels with a stope width less than 1.8 m have spans that exceed the standard (Dunn, 2004).

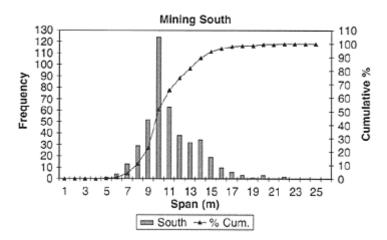
4.3 FOG dimension

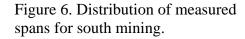
Table 4 summarises the statistical data for the FOG dimension (Dunn, 2004). There does not seem to be a marked difference for the different mining categories. This information can be used to infer the sizes of

Period	Mean	Std. Dev	Max.	Min.	Fit
All: 1991–2001	15.4	7.05	40	4	Beta General
All: 1995–2001	13.8	5.98	32	4	Beta General
North: 1991–2001	14.1	7.12	30	4	Extreme Value

Table 3. Summary of statistical data for pillar spans.

North: 1995–2001	13.0	6.32	24	4	Extreme Value
South: 1991–2001	16.4	6.71	40	7	Normal
South: 1995–2001	15.2	6.21	24	7	Extreme Value
CG: 1991–2001	16.5	7.23	32	7	Extreme Value
CG: 1995–2001	14.2	6.65	32	8	Exponential





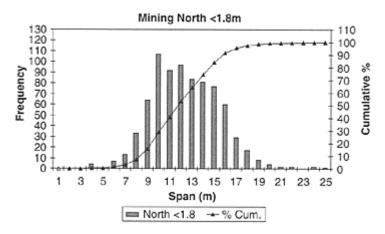


Figure 7. Distribution of measured spans for north mining at stoping widths less than 1.8 m.

domes and the spacing of joints or veins that could form the boundaries of large collapses. Figures 9 and 10 are examples indicating the cumulative distribution and the @RISK fitted distribution for FOG Dimension.

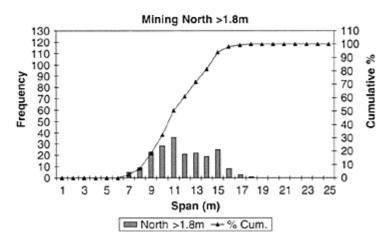
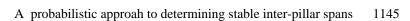


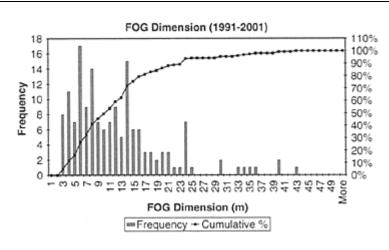
Figure 8. Distribution of measured spans for north mining at stoping widths exceeding 1.8 m.

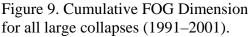
Period	Mean	Std Dev.	Max.	Min	Fit
All: 1991–2001	12.0	8.61	43	1	Pearson5
All: 1995–2001	11.3	7.97	43	1	Inverse Gaussian
North: 1991–2001	12.0	11.67	40	1	Pearson5
North: 1995–2001	10.5	7.84	40	1	Inverse Gaussian
South: 1991–2001	11.7	7.76	43	1	Log Logistic
South: 1995–2001	11.2	6.96	43	1	Extreme Value

Table 4. Summary of FOG dimension statistical data.

CG: 1991–2001	12.5	6.69	40	3	Extreme Value
CG: 1995–2001	12.7	7.00	40	4	Extreme Value







4.4 Cumulative 95% level

The 95% cumulative level was determined for each category and parameter and shown in Table 5 (Dunn, 2004). This is the level below which 95% of the values

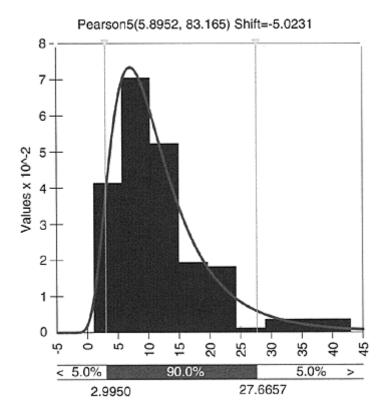


Figure 10. FOG Dimension (m) distribution for 1991–2001 (x-axis indicates the FOG dimension).

Period	Fallout height	Span	FOG Dimension
All: 1991–2001	3.28	28.8	27.7
All: 1995–2001	2.78	25.0	26.8
North: 1991–2001	3.33	27.4	30.9
North: 1995–2001	3.20	24.8	25.8
South: 1991–2001	2.87	27.4	24.9

Table 5. Summary of 95% cumulative level for different parameters.

South: 1995–2001	3.25	26	24.2
CG: 1991–2001	3.51	30.0	25
CG: 1995–2001	2.73	27.5	26

will fall. This approach is commonly used in the design of support in the South African gold mining industry (Daehnke et al, 1998). In addition, the 95% confidence level can be determined from the mean and standard deviation of a distribution. This would be the interval between which there is a 95% certaintity of finding a value.

5 J-BLOCK MODELLING

J-Block is a programme, which allows you to evaluate the potential for gravity driven falls of rock. Geological information is input into the programme, which makes use of statistical methods to simulate (Esterhuizen, 1996) blocks in the hangingwall and identifies potentially unstable keyblocks.

Normally this programme is used to evaluate small key blocks and the performance of normal support systems in controlling key blocks. An attempt was made to simulate large wedge and dome collapses between pillars by using the FOG Dimension and Fallout Height data as a guideline in determining the geological input parameters for J-Block.

The initial results were surprising with extremely low probabilities of failure and occurrences of large keyblocks. This aspect needs further investigation in terms of the geological input parameters.

6 GEOTECHNICAL RELATIONSHIPS

A plan of large collapses was superimposed on the Tau Lekoa geological model. The majority of large collapses occur within the Main channel area, which has the thickest channel widths and a Unit I lava hangingwall. This channel is flanked by major fault structures; the Schoonspruit to the west, and the New Year and Goedgenoeg faults to the east (Biddulph, 1999). Approximately 50% of all mining on Tau Lekoa has taken place in the Main channel and about 80% of large collapses have occurred within the Main channel.

It has been documented that there is a considerable variation in lava strength. Specific areas where poor ground conditions and doming prevail have also been identified. The presence of numerous discontinuities within the immediate hangingwall and a weak altered layer above the VCR in some areas also contributes to ground control problems. Further delineation and quantification of these conditions are required.

Ground control problems are also associated with dip and strike orientated rolls in the reef that result in variable channel widths. These features are especially prominent in slope and reworked channel areas, and result in considerable amounts of external waste (Fourie, 2000).

Steep dipping joints striking north-south and east– west have been identified. Additional to the joint sets are prominent quartz and calcite veins. Many of the large collapses have been associated with prominent, relatively flat dipping veins.

7 DISCUSSION

Falls of ground on Tau Lekoa Mine can be characterised into localised incidents related to a poor quality rock mass, jointing, stress induced fracturing and small scale doming. Larger collapses are related to dome structures and wedges formed by veins and joints.

Smaller falls of ground should be adequately supported by the current in-stope support standards. The crush pillars in conjunction with the in-stope support should control larger collapses. A significant number of collapses have relatively small dimensions and occurred at spans less than the required standards. These incidents should have been prevented by the in-stope support and the possible reasons for this failure are:

- Support not installed within standard.
- Support deteriorates over time and fails as the majority of collapses occurred in back areas or after stoping has stopped.
- A combination of the previous two factors.

Statistically, the correlation between fallout thickness and span is quite poor with a high degree of scatter. This empirical relationship probably does not adequately reflect the underlying geological conditions associated with the larger collapses and the influence of poor quality in-stope support cannot be isolated.

Although this relationship shows a poor statistical correlation, the current pillar layout has been reasonably successful in controlling large collapses. Since 1997 there has been a decrease in the number of large collapses (Figure 11). If the number of collapses are normalised against the square metres mined, it can be seen that there has been an overall downward trend, even though 1996 and 1997 were problematic. This seems to be associated with several large collapses in a specific mining area where the standards in terms of spans were particularly poor.

A careful scrutiny of the database reveals that there are several aspects needing further investigation. Smaller collapses in between pillars spaced to standard must be investigated in detail and should possibly be excluded from the database. The large collapses must also be investigated in more detail with specific attention being given to the contributing geological features.

Several interesting geotechnical relationships have been observed. For smaller falls of ground, areas prone to small scale doming and poor ground conditions must be identified and quantified. Currently, a rock mass rating system is being used (Hungwe & Dunn, 2003).

The majority of large collapses occurred in the Main channel areas. At this stage, there is no conclusive

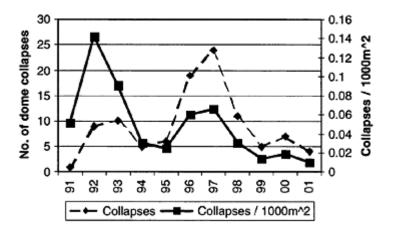


Figure 11. Large collapses per year and normalised against production (Dunn, 2003).

reason or model for this and several theories have been proposed. The distribution of prominent flat dipping veins, joints and possible tensional stresses may all contribute to the formation of dome structures. Lava flows also contribute to the formation of discontinuities within the hangingwall. The identification of the Main channel as a problem area is significant and it is possible that larger spans could be used in other areas (Dunn, 2003).

Although there has been some success with the current pillar layout, there are cases when the design may be inappropriate. In some areas, it may be possible to mine without mid-panel pillars. In certain cases, panel face lengths have been increased to 40 m without any major problems. The cutting of regular mid-panel pillars has a substantial impact on mining efficiencies and their elimination would result in productivity gains.

The distribution, orientation and spacing of various joint sets could assist in the identification of potential wedges. This could be achieved by detailed joint mapping of raises and possibly pre-developed strike gullies. This information would allow probabilistic analyses using programmes such as J-Block and the development of area specific pillar layouts.

The lack of adherence to span standards is a cause for concern. In terms of the high degree of non-adherence, possibly larger collapses could be expected. However, for a collapse to occur, the bounding features of a wedge or dome must fall between pillars and the in-stope must be unable to cope with the demand to control the hangingwall. In some cases undersized pillars would be unable to control the hangingwall, resulting in large effective spans.

This could indicate that the associated risk is not as high as perceived. By defining the distributions of discontinuities, spans and pillar width to height ratios, it may be possible to evaluate the risk of collapses by means of probabilistic methods such as Monte Carlo simulation and J-Block modelling. The work outlined in this paper is the first step

towards this approach with @RISK distributions having been defined for a number of parameters.

Further work is required in terms of filtering out outliers, such as small and large collapses, and conducting a detailed study on possible relationships. Work is also required in terms of defining relationships and conducting a statistical analysis of large collapses in different geological zones. Another study would be to separate the database into standard and substandard categories and conduct a similar analysis.

8 CONCLUSIONS

The current Tau Lekoa Mine Stope Support Strategy has been applied with some success. The design methodology has some weaknesses but has provided useful insight and assisted in identifying areas that should receive greater attention. There is potential to improve on the current design to benefit safety and productivity at Tau Lekoa Mine. The elimination of mid-panel pillars and increasing of pillar spans in some areas would have substantial production benefits (Dunn, 2003).

Optimisation of the current support design and design approach can only be achieved by understanding the geological and geotechnical environment. Some work has already been done but more is required in terms of understanding dome/wedge formation and distribution on Tau Lekoa Mine.

To achieve this all possible relationships must be explored with the aim of defining and quantifying different parameters. Detailed statistical descriptions of the various parameters are required in order to apply probabilistic methods when assessing the risk of wedge or dome collapses.

The focus should be on identifying domes and wedges, and applying probabilistic methods to assess the risk. The identification of areas where small-scale ground control problems will be experienced can be achieved through rock mass ratings.

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Backfill at Sons of Gwalia Mine

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ABSTRACT: Recent underground mining of the Sons of Gwalia Mine at Leonora was carried out from 1999 to 2003. Many of the Sons of Gwalia Mine extensive old workings were unfilled from earlier underground operations, which led to relaxed and collapsed ground conditions, making mining particularly difficult. Due to the range of mining methods used, the low dip of stope walls as well as the need to "prefill" some existing voids, backfill was critical to controlling the rock mass response to mining and hence the success of the operation. The backfill system developed has needed to be flexible and cost effective, with minimal capital expenditure. After reviewing all backfill options, a cemented rockfill system using development mullock was selected. This paper will present the evolution of the backfill system at Sons of Gwalia, including the many challenges encountered, to produce a backfill product that accommodated the shallow dipping stope hanging walls, speed of fill requirements, varying consistency of waste rock supply and cost effectiveness.

1 INTRODUCTION

The Sons of Gwalia Mine is located 235 km north of Kalgoorlie in the Goldfields of Western Australia. Mining on the site first started in 1896 and was closed in 1964 with the deepest workings at 1080 m. Mining was carried out on four lodes, the Footwall Lode, Hanging Wall Lode, Main Lode and South Gwalia Series. The various lodes have strike lengths of up to 500 m and widths varying from 2 to 8 m and dip between 30° to 45°. The South Gwalia series is a thickening of the ore at the southern end of mineralisation. Mining was narrow vein room and pillar or shrinkage stoping, with the majority of stopes left open, which has resulted in delamination and loosening of the Hangingwall Schist where backfill was used it comprised general waste or battery sands. The recent development, open pit and historic workings are shown in Figure 1.

The mine was reopened as a large-scale open pit operation in 1984. In 1998, a decline was developed for trial underground mining of the Hangingwall stopes. The open pit

closed in early 1999 at a depth of 280 m and the underground operation continued at an annual production rate of 600000 tonnes at 4 g/t until 2003.

Recent mining methods of the lodes comprised extracting the vein on a primary/secondary sequence

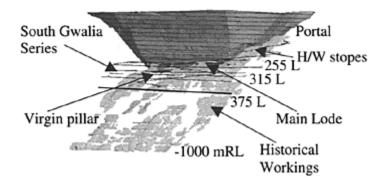


Figure 1. Long section through mine.

along strike and by mining the ore halo around the old workings. Despite the relaxation of ground, all development was in the Hangingwall due to the very poor footwall schist ground conditions. Ore on the Hangingwall of the stopes was not accessible due to the extensive relaxation and the footwall ore was stripped into the void. Any Hangingwall fall off from the Hangingwall was considered a bonus. Backfilling of Main Lode was mostly dry waste rock.

South Gwalia Series was extracted as a massive lowgrade zone using transverse open stoping techniques. The poor ground conditions due to the extensive delamination zones and existing voids, created difficult mining conditions.

A key aspect of the design of this system is economics. High mining costs are experienced due to the problems associated with the old workings and the subsequent delaminated zones.

2 DEVELOPMENT OF BACKFILL

During the planning stages of the re-commencement of underground mining at Gwalia, geotechnical studies identified that due to the extensive voids, the anticipated high *in situ* stress and generally poor ground conditions, backfill would be required immediately following extraction of a stope to minimise the hangingwall span and provide a working platform for the stope above (Golder, 1995). The backfill would also reduce further deterioration of the hangingwall and therefore also the decline.

2.1 Investigations

All backfill methods were reviewed for consideration. Hydraulic fill was excluded due to the fine nature of mine tailings, low recoveries of classified tailings would be available for hydraulic fill as well as the risk of barricade failure with personnel working close to stoping areas.

Paste fill was considered but the results of the laboratory study on mine tailings showed extraordinarily low strength gains. The laboratory testing indicated that irregular strength gains could be achieved with 25% rock addition rate and that at 50% addition rate of rock, the strengths were approximately doubled. However, the cement addition rates required to achieve the target strengths were still high. Another factor that led to the exclusion of paste was the high capital cost in light of the uncertainties associated with the mining conditions and proposed mining method.

For these reasons, a low capital cost rock fill system was designed. Waste rock from development headings, supplemented with backhauled surface waste was delivered to a "mixing" bay, where it was prepared in 60 tonne batches. Cement slurry was delivered by agitruck from a batch plant in Leonora in 3 m3 batches to the cuddy. A bogger then mixed the rock and cement slurry before delivering the cemented rock fill to the stope. The development waste generally had a high moisture content and hence the mix design had a cement to water ratio of 1.2. The 3 m³ of slurry per batch was used to give a 4% cement addition rate and to prevent spillage of the slurry from agitruck.

3 EXPERIENCE

3.1 Early cemented rock fill of Hanging Wall stopes

Where relatively steep stopes were backfilled, the system was demonstrated to be very successful with minimal backfill dilution. However, tight filling the hangingwall was not always achieved, as the angle of repose of loosely placed rock was 41°, and similar to the dip of the orebody. The high angle of repose was due to platey nature of the waste and also due to a lack of quality control of the mix. Generally, filling of secondary or isolated stopes proved more difficult to fully support the hanging wall. This lead to a focus on using the cement slurry and fines to produce a flowable rockfill that could achieve flatter angles of repose.

Uphole mining was successfully carried out beneath one backfilled 8 m wide stope with minimal fill dilution from the backs.

3.2 Filling of Main Lode Virgin Pillar stopes

These stopes were filled in much the same way as the earlier Hanging Wall stopes and as a result were not successful in obtaining a tight fill, as shown in Figure 2. As a result, amongst other contributing factors, large movement of the open pit hangingwall was experienced and a number of development drives and stopes were lost.

A rocky paste was trialed in one stope, which could not be filled using conventional rock fill due to a lack of access. A rise was developed from the level above to dump the paste into. The paste was prepared from a mixture of tailings, minus 14 mm aggregate and cement slurry at 4% addition. Difficulties were encountered with loading the tailings mixture into the agitator as it tended to "ball" in the agitrcuk, preventing adequate mixing.

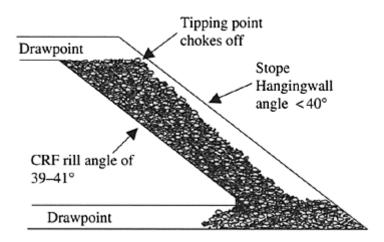


Figure 2. Section through a stope showing rill angles of backfill compared to stope hangingwall angle.

3.3 Modified CRF system

The agitruck delivered cement slurry was useful during early stages of underground mining at Gwalia, to assist in gaining an understanding of backfill at minimal capital cost. However, as mining moved progressively away from the mine portals, the cost of supplying the cement increased dramatically. For this reason, a surface cement mixing plant was installed (Figure 3), and included a 60 tonne silo and agitator mixer. A borehole conveyed the slurry directly to the mixing bay has two steel 100 mm pipes in a 250 mm borehole.

Due to the reduction in development waste generated, a 2.4-m raise bore waste pass was developed to 275 m below surface, close to the slurry hole and mixing bay, as shown in Figure 4. A bogger continued to be used to mix and haul the cemented waste to the stope.

3.4 South Gwalia stope design

The South Gwalia Series stope shape was designed so that tight fill can be achieved with CRF with rill

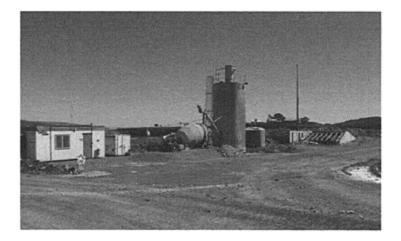


Figure 3. Surface facilities showing slurry plant and grizzly above waste pass.

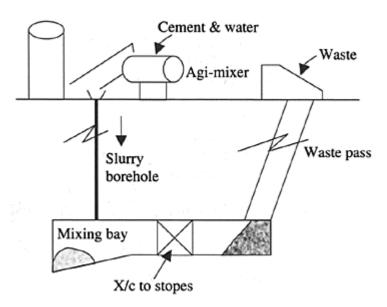
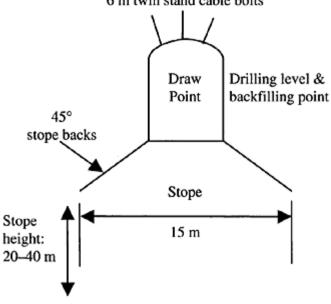


Figure 4. Schematic layout of backfill system.

angles equal or less than the rill angle of waste rock. The stope backs were designed at 45° as shown in Figure 5.

The primary difficulty encountered with backfilling was the steep rill angles of the fill, which were often greater than 50°. This problem related to variations in fines in the waste rock and the water content of the cement slurry, both of which affected the flowability of the mix. While at some operations it may be advantageous to have such a steep angle, at Gwalia it resulted in a lack of support to the hanging wall and potentially, insufficient water available for cement hydration. Investigations identified the following contributing factors:

- High summer temperatures caused cement-mixing water to be over 50°, causing early hydration of the cement and possibly loss of water through evaporation. The response by operators was to reduce the amount of rock being added to facilitate mixing, which increases cement consumption and costs. Providing cool water proved to be uneconomical. Therefore, the water content was increased, which increased the water to cement ratio. While this theoretically would lower the strength, the overall fill mass has higher strength because the particles are coated with cement.
- The waste material being used had a moderately high strength (80 to 100 MPa) but suffered from high degrees of attrition. This resulted in an increase in fines in the waste rock by over 20% during passage in the waste fill pass, as shown in Figure 6. The increased fines content increased the surface area that is to be coated, and reduced the flowability of the mixture.
- Variations in the nature of surface waste material used, results in changing rockfill properties, and



6 m twin stand cable bolts

Figure 5. Stope profile.

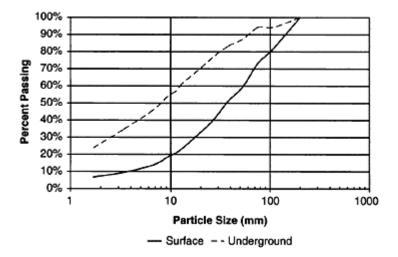


Figure 6. Variation in fines before and after it has passed down the rise.

have highlighted the importance of the loader operator to correctly identify suitable materials.

These issues highlight the importance of daily monitoring and quality control of the backfill mix design. While the system required little capital cost, all controls (except water addition to the slurry) were based on observations, which can differ between operators. For this reason, significant efforts were made on training and providing prescriptive measures (ie. having a fixed number of buckets of waste) and daily inspections.

3.5 Other backflll issues

Excessive over break above the 45° section of the stope back, (Figure 5), due to the delaminating along foliation planes and blasting practices, was common, making filling to the hangingwall difficult. A number of solutions were attempted to properly fill the void, including:

- Washing waste rock into Hangingwall gap with water;
- Pushing the fill with the loader into the fall off zone; and
- Pumping a weak concrete mix into the void;

All of these were ineffective or expensive. The most effective means was a combination of smooth wall blasting of the stope backs, supplemented with increased cable bolting and preparing a wetter rockfill.

All secondary stopes were waste filled directly with waste development where possible and supplemented with surface waste rock from the waste pass.

The success of the producing a flowable cemented rock fill is shown in Figure 7, in this case, a rill angle of 32°. Stopes with hangingwall angles as low as 35° were routinely

tight filled and on one occasion, a 25° stope was successfully tight filled (Figure 8). To ensure this success, considerable effort was made to



Figure 7. Backfill rill angle at 32°



Figure 8. The photo shows tight backfill with a stope hanging wall angle of 25° .

monitor the mix quality and also to train, on an on going basis, the Mine Foreman, shiftbosses and operators on the importance of maintaining fill quality.

3.6 Mix design and test work

To assist in optimising the cost of backfilling, strength testing has been carried. As there is no adequate facilities close to site, a 50 tonne hydraulic press in Leonora was modified to test samples. The samples were prepared underground by scalping fragments over 100 mm in diameter before casting the samples in 300 mm diameter and 600 mm length PVC pipe. The cylinders were cured underground before testing (Figure 9). This method partially overcame the problem of aggregate size affecting strength test results, which is commonly encountered with smaller size samples.

3.7 Development through backflll

Due to limited and difficult access, development through previously backfilled stopes was often

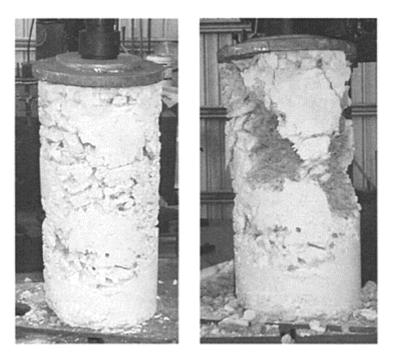


Figure 9. Samples before and after testing.

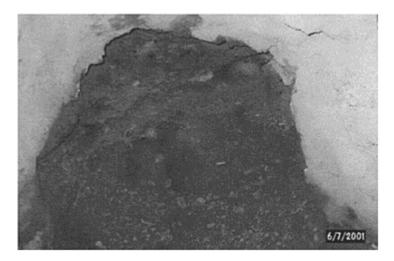


Figure 10. Development through backfill.

required. In some areas, the fill could be excavated by the loader but generally, drill and blast was required. Blasting would involve a round of widely spaced holes with friction bolts inserted to keep the holes open. The blast holes were then charged, with bolts remaining in the holes, with low density ANFO. This achieved good profiles, which would then be covered with shotcrete as a protective layer to prevent unravelling over time.

Developing through the backfill confirmed that the quality of fill being placed was generally satisfactory, although segregation, and hence variable strength, was common. An example is shown in Figure 10, where the toe of the face contained approximately one metre of low strength fill and generally large fragments. Above this layer, the quality of mix greatly improved with generally well-graded material with about 10% to 30% coarse material.



Figure 11. Secondary stope failure.

4 EFFECTIVENESS OF BACKFILLED STOPES

The mining method employed in the South Gwalia Series and "Virgin Pillar" was a sequence of primary/ secondary stopes. Calculations and experience showed that hanging wall spans greater than 15 to 20 m along strike would result in wall failure. Secondary stopes had the increased risk of failure due to increased span due to lack of tight primary stope backfilling and also from disturbance to the ground conditions through blasting and relaxation.

Extraction of the "Virgin Pillar" is a good example of poor primary stope filling. Almost all the secondary stopes mined resulted in dilution from large scale hanging wall failures. Figure 11 shows a secondary stope hanging wall failure of almost 1 m (20% dilution), the exposed bolt plates showing the design hanging wall contact.

The general experience demonstrated that the unfilled stopes from mining pre-1960, resulted in extensive loosening and relaxation of the rock mass that was hard to contain. Even with backfilling of recent stoping, where effective tightfilling resulted, relaxation occur to the surrounding area.

The effect on the decline where tightfilling of stopes was not achieved caused instability problems with monitoring showing that over time rehabilitation was required.

Cable bolting (6 m twin strand, bulbed) was used to supplement support of the development where loose fractures were encountered and provided safe access. In addition, cable bolting of the backs of stopes was used in the wider South Gwalia Series but due to the lack of fill and or tight fill relaxation allowed the foliation planes to open causing grout loss.

Successful secondary stope extraction has been limited due to a number of reasons. These are mainly due to extensive relaxation due to the old workings. The 245 ML24

stope was a successful secondary stope. Exposure of the backfill on both sides confirmed tight fill against a hanging wall dipping at approximately 45°.

5 CONCLUSION

Mining at the Leonora operation required flexible and adaptable approach to accommodate the unique conditions encountered. The backfill system evolved to meet the changing mining methods and sequencing options that have been trialed. The cost of backfilling was optimised with time and capital costs invested in a progressive fashion to limit exposure.

The effectiveness of the backfill in limiting further ground movement assisted in preventing further deterioration of the main access in the hangingwall but proved limited in providing successful secondary stope extraction.

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Measurement and prediction of internal stresses in an underground opening during its filling with cemented fill

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ABSTRACT: Knowledge of the internal stresses of cemented fill used in underground mines is very important with respect to ground control. The stope of a Canadian gold mine was instrumented with earth pressure cells to measure the stresses within the pastefill during and after filling. The results indicate that the stresses increase during backfilling but tend to decrease after the completion of backfilling. The Longitudinal stresses were observed to be the highest, while the vertical and transverse stresses were lower and roughly equivalent. Two new 3D models are proposed to predict the internal stresses of pastefill and the pressure on barricades as a function of the filled height. These 3D models have been extended to take into account the time factor in the development of stresses within pastefill. Comparison of the model responses with experimental data indicates that the models predict the stresses within the pastefill and the pressure on the barricade with reasonable accuracy.

1 INTRODUCTION

The recovery of underground hard rock ore bodies often involves the use of mine fills. The type of mine fill depends on the mining methods and sequences. Where later recovery is dependent on stability of exposures of earlier placed fill, cement and/or cementitious materials are added to the fill. Once such fill type is pastefill which is in fact becoming a standard practice in Canada (e.g. Landriault 1995, Landriault & Tenbergen 1995, Naylor et al. 1997). However, the use of pastefill to maintain ground stability involves some difficulties related to the complexities of its behaviour. These complexities are due to the continuous evolution of the properties of cemented fill during placement, consolidation, and hardening due to the hydration of binder agents.

Despite recent work conducted on cemented fills (e.g. Hassani & Archibald 1998, Benzaazoua et al. 2002, Bernier et al. 1999, Benzaazoua & Belem 2000, Belem et al. 2002) many questions remain concerning the stability analysis of a stope filled with pastefill. Indeed, after backfilling with cemented fill the structural integrity of a stope can be threatened by several macroscopic factors (exclusive of the hydration process) which influence the mechanical strength of the pastefill. These factors include: compression and consolidation of the pastefill, the volume and geometry of the stope, the stress distribution within the backfill and between the backfill and the stope, wall convergence, shrinkage, and the effect of arching within the pastefill. Consequently, an understanding of these various factors of influence is necessary to provide more efficient means of ground control. Indeed, knowledge of the magnitude of pressures on barricades will allow for better planning of mining sequences. Additionally, the knowledge of the stress fields within pastefill will facilitate analysis of its stability when one of its faces will be exposed or when an access gallery to a new stope is excavated through the pastefill mass.

The objective of this study was to follow the evolution of pressures developed in cemented fill during placement and consolidation. There is very little data or documentation regarding *in situ* measurement of the mechanical properties of pastefills. However, some work has carried out in this direction (Gay et al. 1988, Ouellet & Servant 2000, Been et al. 2002, Le Roux 2004). Work concerning the instrumentation of pastefill has been conducted by (Corson 1971, Hassani et al. 1998, Hassani 1999, Rankine et al. 2001, Revell 2003). A common point of these experimental studies is that the results are limited to the subject mines.

In this paper, we first briefly describe the Doyon Gold Mine where a selected trial stope was instrumented and backfilled. The influence of the curing time of the pastefill and the height of fill on the measured pressures will be discussed. Based on the results, analytical models will be proposed to predict the pressures developed in pastefill as well as the pressure on barricades during and after backfilling.

2 DOYON GOLD MINE

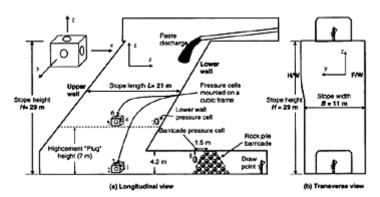
The Doyon Gold Mine, property of Cambior Inc., is located approximately 40 km east of the city of Rouyn-Noranda and has been in operation since the beginning of the 1980's. To-date, about 25,600,000 metric tons of gold ore have been extracted from the mine. Doyon Gold Mine uses the open stoping mining method in conjunction with post placed paste backfill since the late 1990's. The stoping extends to a maximum depth of 800 m over an area of 1200 m× 600 m. The long-hole method is the only mining method used at the site. Since the ore body consists of narrow veins of quartz-pyrite-tourmaline of different width (0.1 m to 1.2 m), the stope dimensions are not the same and vary from 3 m×23 m in plan and 22 m high to 12 m×21 min plan and 30 m high.

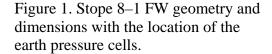
Currently, up to approximately 140 stopes are backfilled per year at the mine (roughly 1700 tons of pastefill are placed per day). The pastefill is transported by gravity using 15cm diameter pipes in a staircase network (Harvey 2004). The stope backfilling method consists of pouring first the "plug" (high cement content) up to 3 m behind the draw point followed by a 1-day curing time. After that the rest of the stope is filled ("residual fill").

3 DESCRIPTION OF THE INSTRUMENTATION

3.1 Location of measurement points

To follow the evolution of the pressures developed in the pastefill, two trial stopes were selected and instrumented each with eight pressure cells but only the Stope 8–1 FW of the Doyon Gold Mine is concerned in this paper. The pressure cells were placed at four locations within the stope: the floor of the stope, at the plug/residual fill interface, on the lower wall, and on the barricade. Figure 1 shows the geometry and the dimensions of the stope, which has an average width,





B, of 11 m (along the transverse axis, *y*) a length, *L*, of 21 m (along the longitudinal axis, *x*) and a height, *H*, of 29 m (along the vertical axis, *z*). The trial stope is oriented at an azimuth of 90° and a dip of 90° and is located at a depth of 450 m in a zone where there were no more production sequences planned temporarely. Consequently, the initial stress field was relatively stable and the access to the stope was safe. The pastefill pore pressures were not monitored in this study because any drainage of free water at the rock pile-shotcrete barricade was observed (no build-up of the total earth pressures).

The development of pressures in the pastefill was measured in three dimensions corresponding to the *x*, *y* and *z* axes at two locations (on the floor of the stope and at the plug/residual fill interface) as shown on Figure 1a. A single pressure cell was placed along each axis at these locations, 3 pressure cells on the floor of the stope (cells 1, 2 and 3) and 3 pressure cells at the plug/residual fill interface (cells 4, 5 and 6). A single pressure cell was placed on the lower wall (cell 7) at the same longitudinal axis as cell 4 and another pressure cell was placed at mid-height of the draw point of the stope at 1.5 m from the barricade (cell 8) on a longitudinal axis (Fig. 1a).

3.2 Experimental devices for pressure measurement

3.2.1 Earth pressure cells used

The earth pressure cells were model TPC by RocTest, which were considered appropriate for this type of measurement as suggested by Weller & Kulhawy (1982). The model TPC consists of a sealed distribution pad composed of two 230-mm diameter circular plates welded together around their peripheries and filled with de-aired oil. These pads were connected via lengths of steel tubes to vibrating wire pressure transducers and 30-meterlong cables. This model has a built-in 3 K Ω thermistor which allows temperature readings from -55° C to $+85^{\circ}$ C (see Fig. 2). The cell capacity was 750 kPa with an accuracy of $\pm 0.5\%$ of the full scale (i.e ± 3.75 kPa). The cells were capable

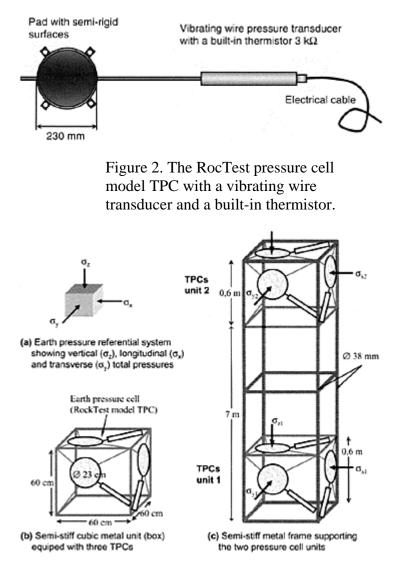


Figure 3. Experimental device for the instrumentation of the paste backfill mass.

of operating at up to twice the rated capacity with reduced accuracy. The readings of the total pressure were taken using a model MB-6T portable read-out unit.

3.2.2 Device for the pastefill mass

Figure 3a shows the orientation of the three total pressures, σ_x , σ_y , σ_z , which were measured in Stope 8–1 FW. Two semi-stiff cubic metal boxes (60 cm) were manufactured. Three pressure cells were mounted on three faces of each box (Fig. 3b). This type of arrangement has been employed in the past for similar measurements (Hassani et al. 1998, Hassani 1999). To maintain alignment the boxes were assembled on a metal semi-stiff frame of 7.6 m high (Fig. 3c).

The frame is then placed in the stope by means of an in-house manufactured trolley mounted on two wheels and a mechanism of pulley and cords. The trolley supporting the frame is firstly thorough to 3 m inside the stope and then the frame is raised by pulling at the same time on two cords, one from the draw point and the other one from the upper gallery. In the absence of angle indicators on the device, bands of fluorescent painting on the top of the frame allow its visual upright positioning in the stope. After its installation,

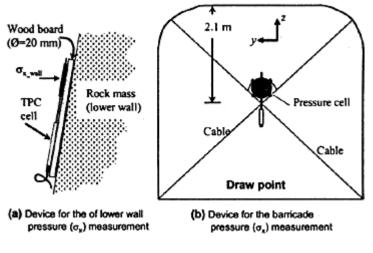


Figure 4. Experimental devices to instrument (a) the stope lower wall and (b) the draw point in front of the barricade.

it was observed that the device had a slight deviation from vertical estimated at 5 degrees. This deviation was neglected in our interpretations.

3.2.3 Device for the foot wall

The cell which was mounted on the lower wall of the stope was fixed to a 20-mm-thick wood board based on the recommendations of Weller & Kulhawy (1982). The corresponding aspect ratio (ratio of the cell diameter to the board thickness) was 11.5 (Fig. 4a). The device was easier to set up on the wall and prevented direct contact between the cell and the rock face. A similar device has been used by Yang et al. (1998) for the measurement of the pressures developed on concrete box culverts under highway embankments.

3.2.4 Device for the barricade

The TPC cell near the barricade was installed vertically at the intersection of two 6-mmdiameter steel retaining cables (Fig. 4b). In such a configuration the cell could not undergo rotation, but could possibly undergo longitudinal displacement of 2 to 4 cm (Harvey 2004). Such a displacement could cause a slight underestimate of the pressure on the barricade.

4 BACKFILLING OF THE STOPE

4.1 Paste backfill mix design

The tailings from the concentrator at the Doyon Gold Mine were used for the pastefill. At the outlet of the thickener the pulp is 60% solids by mass. It is then stripped of cyanide and routed to disc-type filters where it is dewatered to 80% solids. The tailings pulp is then mixed with the binder agents and water to create cemented paste. The mix used at Doyon Gold Mine is 7% by mass of Portland cement for the "plug" which is used to fill the lower portion of the stope (7 m in this case), and 3% by mass of binder (30% Portland

Table 1. Variation of the Doyon Gold Mine pastefill moisture content, w(%) and solids concentration by mass, $C_w(\%)$ in the course of curing time.

Plug			Residual f	ill
Curing time	w (%)	C _w (%)	w (%)	C _w (%)
0-day	42.9	70	42.9	70
7-day	41.6	70.6	42.7	70.1
14-day	41.0	70.9	41.4	70.7

28-day 40.0 71.5	40.6 71.1
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cement and 70% slag) for the "residual fill" used to fill the remainder of the stope (22 m in this case).

Because of the clayey nature of the Doyon Gold Mine tailings (more than 40% clay fraction) its Specific Gravity is of 2.73 and the average solids concentration of the resulting pastefill is 70% by mass with an average slump of 210 mm (moisture content of 42.9%). This low solids concentration is due to pastefill low value of bulk density (1.8 tm⁻³) and to the fact that Doyon Gold Mine tailings contain about 50% fines (grains diameter $<20\mu$ m) while 15% of fines would have been optimal for the needs of the pastefill transport through pipes via gravity (Landriault et al. 1997). Also the clayey nature of this cemented fill exhibits its strong water retention capacity.

The bulk unit weight (γ) of the Doyon pastefill is of 18 kN/m3 (degree of saturation S_r =100%) and its dry unit weight (γ_d) is of 12.6 kN/m3. The initial void ratio (e_0) and corresponding initial porosity (n_0) of the pastefill is 1.18 and 0.54 respectively for the plug (7% Portland cement) and 1.17 and 0.54 for the residual fill (3% binder). Table 1 presents the variation of the moisture content and the solids concentration of the Doyon Gold Mine pastefill after 7-, 14- and 28-day curing time.

The observed mechanical strength are rather weak and the average 7-day compressive strength is about 170 kPa for the plug (7% Portland cement) and of 130 kPa for the residual fill (3% binder agent).

4.2 Stope backfilling with pasteflll

Backfilling of Stope 8–1 FW with pastefill began four weeks following the last mining sequence (wall convergence was assumed to be complete by this time) and was carried out in three sequences (Fig. 5). The first sequence (3045 tons of pastefill) consisted of the pouring of the plug (h=7.3 m) and lasted 44 hours (\approx 2 days) followed by a curing period of 94 hours (\approx 4 days).

The second sequence (10,339 tons of pastefill) consisted of the placement of 18 m of residual fill and lasted 190 hours (\approx 8 days) followed by a curing period

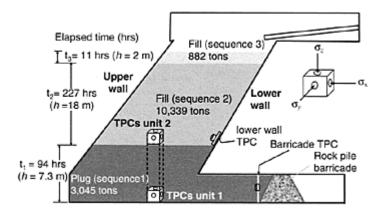


Figure 5. The filling of Stope 8–1 FW with cemented paste backfill in three sequences.

of 37 hours. The third sequence (882 tons of CPB) consisted of the completion of the residual fill by an additional 2 m, the remaining 2 m of the stope were left empty. The total duration of filling including the curing periods was 356 hours (\approx 15 days) and a total of 14,266 tons of pastefill were placed (Harvey 2004).

5 RESULT OF PRESSURE MEASUREMENTS

The pressure readouts were taken from the start of backfilling until 320 days after the end of backfilling. Due to the geometry of the stope (see Fig. 5), the filled heights (h) were calculated from the pastefill quantities. The duration of filling and the stope volume were obtained from a CMS (Caving Monitoring System) scanning. Figure 6 shows the variation of filled height (h) with respect to the time elapsed since the start of filling.

5.1 Internal pressure in pasteflll during placement

5.1.1 Pressure developed at the floor of the stope

Figure 7 shows the evolution of the vertical (σ_{z1}) , longitudinal (σ_{x1}) and transverse (σ_{y1}) pressures at the floor of the stope during the filling which lasted 15 days including the curing periods. After 4 days the filled height, *h*, was 7.3 m (actual plug height) and at the 15th day the filled height was 27 m. The point of measurement of the longitudinal and transverse pressures was located at elevation, *z*, of 0.3 m and for the vertical one at elevation of 0.6 m.

It can be observed from Figure 7, that during filling the longitudinal pressure (σ_{x1}) was the highest on the floor of the stope (Fig. 7). This pressure reached its maximum value (σ_{x1_max}) of about 150 kPa during the second sequence of filling (after the 10th day). This maximum value is almost twice that of the vertical (σ_{z1}) and transverse (σ_{y1}) pressures which were similar in magnitude ($\sigma_{x1} > \sigma_{y1} \approx_1 \sigma_{z1}$). It should be noted that the 882 tons of pastefill added during the

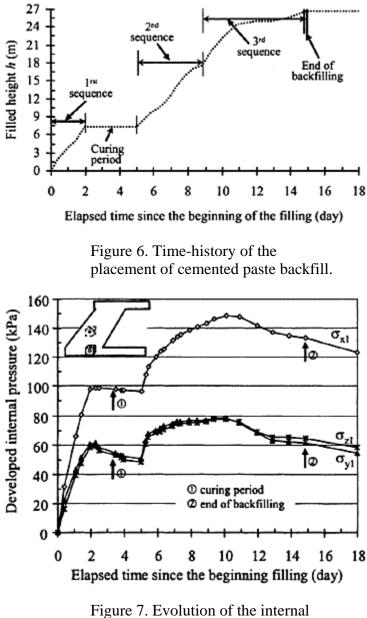


Figure 7. Evolution of the internal pressures of the CPB at the floor of Stope 8–1 FW as a function of elapsed time since the beginning of the filling.

3rd sequence of filling, from the 13th day, had no influence on the internal pressures at the floor, which actually began decreasing. This reduction continued until day 91.

5.1.2 Pressure developed at the plug/fill interface

The TPC cells which were located at elevation 7.6 m for σ_{z2} and at elevation 7.3 m for σ_{x2} and σ_{y2} began recording pressures only after the pastefill rose to that level, some 30 hours after the start of the placement of the residual fill (124 hours from the start of filling). Figure 8 shows the evolution of the vertical (σ_{z2}), longitudinal (σ_{x2}) and transverse (σ_{y2}) pressures at the plug/residual fill interface from the 5th day of filling. Again, it can observed that the longitudinal pressure is the highest of the three measured pressures ($\sigma_{x2}>\sigma_{y2}>\sigma_{z2}$). The maximum value of the longitudinal pressure (σ_{x2}) is about 53 kPa ($\sigma_{y2}=38$ kPa and $\sigma_{z2}=25$ kPa).

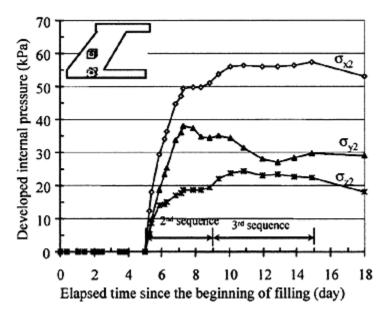


Figure 8. Evolution of the internal pressures of the paste-fill at the elevation 7.6 m (plug/residual fill interface) during the course of the filling of Stope 8–1 FW.

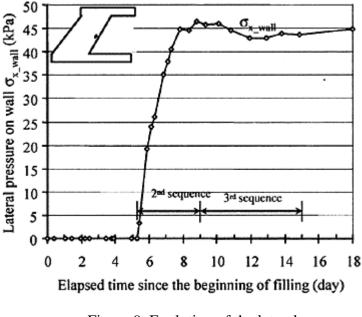


Figure 9. Evolution of the lateral pressure on the lower wall at elevation 7.6 m during the filling of Stope 8–1 FW.

5.2 Lateral pressure on the lower wall

The TPC cell located on the lower wall also began registering pressures 30 hours after the start of place-ment of the residual fill. Figure 9 shows the evolution of the longitudinal pressure (σ_{x_wall}) exerted on the lower wall of Stope 8–1 FW. The readings began the 5th day and the maximum value reached was about 45 kPa.

5.3 Lateral pressure on the barricade

Figure 10 shows the evolution of the longitudinal pressure (σ_{x_b}) exerted on the barricade at mid-height (z'=2.1 m). It can be observed that the maximum pressure exerted on the barricade was about 54 kPa

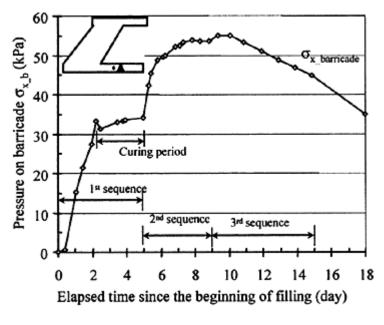


Figure 10. Evolution of the lateral pressure on the barricade at mid-height of the lower gallery (z'=2.1 m) during the course of the filling of Stope 8–1 FW.

and was reached on the 10th day of filling. However, this pressure begins to decrease just after having reached this maximum.

6 DISCUSSION

6.1 Long-term behaviour of pasteflll

The time history of pressure measurements permits observation of both the variation of the developed internal pressures and the effect of hydration of the binder reagents. The pressures on the floor of the stope will become critical when they approach the compressive strength of the pastefill. For example, an increase in the transverse pressure (σ_y) would probably indicate wall convergence. From the point of view of the mine production and safety, it is also important to know when the pressure being exerted on the barricade will be dissipated. Finally, the long-term evolution of the pressures in the pastefill allows estimation of its consolidation characteristics, and the stress redistribution due to the mining at the vicinity of filled stope.

Figure 11 shows the evolution of the internal stresses of the pastefill at elevation 0.6 m as a function of the elapsed time since the beginning of the filling. It can be observed that

after reaching their peak values, all the pressures decrease after the end of the filling (15th day) until approximately the 110th day. Beyond 122 days a continual increase in the longitudinal (σ_{x1}) and vertical (σ_{z1}) pressures until 361 days is observed. On the other hand the transverse pressure (σ_{y1}) increases and then began decreasing. The same tendencies were observed for the pressures (σ_{x2} , σ_{y2} , σ_{z2}) measured at elevation 7.6 m. The increase in the pressures beyond 122 days is probably due to the

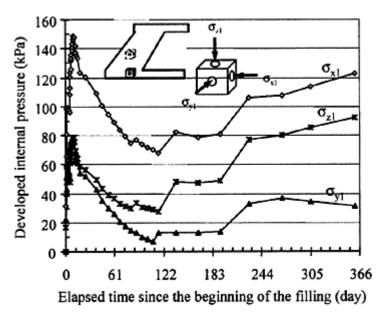


Figure 11. Long-term evolution of the internal pressures of the pastefill at the floor of the stope.

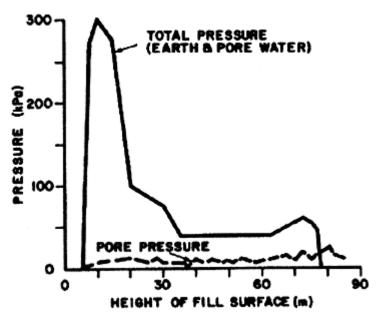


Figure 12. Pressures on barricade as a function of the fill height (After Bridges 2003).

circulation of heavy machines on the top of the fill as well as new mining sequences at the vicinity of the Stope 8–1 FW.

Pore pressure of pastefill is necessary for the calculation of the effective earth pressure, but was not measured in this study Even if the rock pile barricade of the trial stope allows the drainage of free water, any drainage was quantified. Due to its strong water retention capacity the Doyon Gold Mine pastefill remains saturated (moisture contents of about 38%) a long time until 360 days and the drainage period, if any, does not exceed 5 days. Indeed, *in situ* measurements of pore pressure of pastefill on barricade showed that pore pressure is negligible as shown on Figure 12 (Bridges 2003).

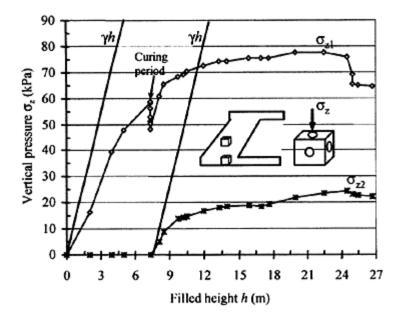


Figure 13. Variation of the internal vertical pressure σ_z of the pastefill as a function of the filled height.

6.2 Effect of filled height on the developed pressure

6.2.1 Vertical pressure

Figure 13 presents the evolution of the vertical pressure (σ_z) at elevation 0.6 m (bottom of the plug) and at elevation 7.6 m (plug/residual fill interface) compared to the theoretical overburden stress o the pastefill (γh). This comparison allows verification of the existence of an arching effect. An arching effect would reduce the magnitude of the vertical pressure at the floor of the stope ($\sigma_z < \gamma h$) which will be compensated by an increase in the longitudinal pressure (σ_x) on the walls of the stope (Aubertin et al. 2003; Li et al. 2003, 2004). The curves thus show that there was an arching effect in the filled stope.

6.2.2 Longitudinal pressure

Figure 14 presents the evolution of the longitudinal pressure at elevations 0.6 m (plug) and 7.6 m (plug/ residual fill interface and lower wall). The longitudinal pressure at the bottom of the stope (σ_{x1}) is more than twice that measured at the plug/residual fill interface (σ_{x2}) as well as at the lower wall (σ_{x_wall}). It is also noted that the pressure exerted on the lower wall (σ_{x_wall}) is slightly lower than that measured on the same axis but at a distance of 3 m (σ_{x2}). The maximum longitudinal pressures σ_{x1} and σ_{x2} were

obtained at a filled height of 22 m while the maximum longitudinal pressure on the lower wall (σ_{x_wall}) was obtained at a filled height of 18 m.

6.2.3 Transverse pressure

Figure 15 presents the evolution of the transverse pressure at elevations 0.6 m (plug) and 7.6 m (plug/ residual fill). This figure shows that the transverse

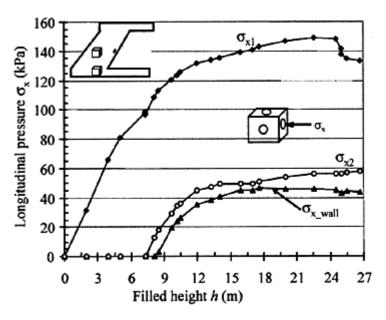


Figure 14. Variation of the internal longitudinal σ_x pressure of the pastefill as a function of the filled height.

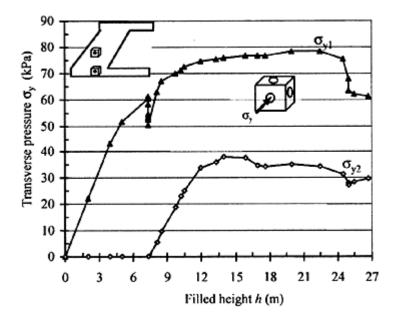


Figure 15. Variation of the internal transversal pressure σ_y of the CPB as a function of the filled height.

pressure at the bottom of the stope (σ_{x1}) was twice that measured at the elevation 7.6 m (σ_{x2}) . The maximum lateral pressure σ_{x1} was obtained at a filled height of 19 m while σ_{x2} reached a maximum value at a filled height of 14 m.

6.2.4 Pressure on the barricade

Figure 16 presents the evolution of the lateral pressure on the barricade ($\sigma_{x_{-}b}$) as a function of the filled height. This pressure increased continuously and reached its maximum at a filled height of 22 m. Beyond this filled height the placement of additional pastefill did not result in an increase in pressure on the barricade, on the contrary the pressure decreased.

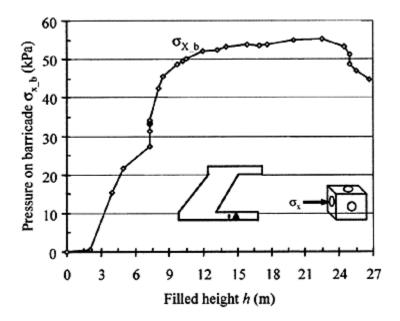


Figure 16. Variation of the lateral pressure σ_{x_b} on the barricade as a function of the filled height.

6.3 Modeling pressures development in the CPB during stope filling

The results of the pressure measurements presented in this paper clearly show that the pressures decreased slightly, by about 8 kPa, (except for the pressure on the barricade, see Fig. 10) during the curing period (between the 2nd and the 5th day). This indicates that the development of the internal pressures may be independent of the hydration of the binder reagent (see Fig. 7). Consequently, the dominant factor appears to be the filled height. Moreover, during pastefill placement it is helpful to know the evolution of the pressures developed within the pastefill based on the filled height. Accordingly, we propose simple 3D models to allow the prediction of both the three-dimensional pressures (σ_{x} , σ_{y} , σ_{z}) developed in the pastefill and the pressure exerted on the barricade (σ_{x_b}) during filling.

6.3.1 Filled height-dependent 3D model to predict the internal stresses of pastefill

According to the results presented, the internal stresses of the CPB increased gradually as a function of the filled height to some maximum values. These stresses then remained relatively constant at the end of the filling. With regard to the bottom of the stope, the longitudinal pressure (σ_{x1}) was almost twice that developed vertically (σ_{z1}) or transversely (σ_{y1}). This type of variation suggests that the pressure at the floor of the stope depends on the unit weight of the pastefill, and more importantly on the dimensions

of the stope, apparently as a result of the arching effect. Thus, the variation of the longitudinal pressure (σ_x) depends on its maximum value (σ_x) max and on the filled height (*h*). This variation of σ_x can be described by an exponential relationship (as proposed by the

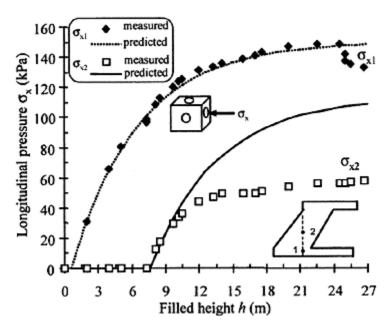


Figure 17. Comparison between experimental data and predicted curves of the longitudinal pressure σ_x at two elevation points (*z*1=0.6 m and *z*2=7.6 m) using Equation 3: γ =18 kN/m³, $H_{\rm m}$ =29 m, *B*=12 m, *L*=21 m.

Marston theory; see McCarthy 1988 and Aubertin et al. 2003), which can be formulated as follows:

$$\sigma_x(h) = \left(\sigma_x\right)_{\max} \left[1 - \exp\left(-\frac{(h-z)}{a}\right)\right] \tag{1}$$

where *a* is a constant of proportionality; *h* is the filled height (m); *z* is the elevation (m); and $h \ge z$.

The maximum longitudinal pressure, $(\sigma_x)_{max}$, depends on the overburden stress of the CPB (γH) and can be estimated by the following relationship:

$$(\sigma_x)_{\max} = \gamma (H_m - z) * \frac{H_m}{3(B+L)}$$
(2)

where γ is the bulk unit weight of the CPB (kN/m³); H_m is the total height of the filled stope (m); *z* is the elevation (m): *z*=0 at the floor of the stope, *z*= H_m at the top of the filled stope; *B* is the stope width; and *L* is the stope length.

Substituting Equation 2 into Equation 1 and assuming that the constant *a* is half of the stope width B (*a*=B/2) leads to the following 3D model:

$$\sigma_x(h) = \frac{\gamma H_m (H_m - z)}{3(B+L)} \left[1 - \exp\left(-\frac{2(h-z)}{B}\right) \right]$$
(3)

where $z \leq h \leq H_m$.

Figure 17 shows the measured longitudinal pressures at the elevations 0.6 m (σ_{x1})—floor of the stope and 7.6 (σ_{x2})—plug/fill interface—compared to the predicted values using Equation 3. Note that this 3D

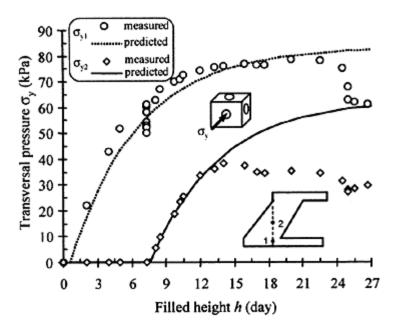


Figure 18. Comparison between experimental data and predicted curves of the transversal pressure σ_y at two elevation points (*z*1=0.6 m and *z*2=7.6 m) using Equation 4: γ =18 kN/m³, H_m =29 m, *B*=12 m, *L*=21 m. model describes the longitudinal pressure at the floor of the stope (σ_{x1}) reasonably well, but at the plug/ residual fill interface (σ_{x2}) is not as accurate.

From the results of pressure measurements presented in this paper (e.g. Fig. 7) one can reasonably assume that the vertical pressure (σ_z) developed in a stope backfilled with pastefill is approximately equal to the developed transverse pressure (σ_y). From this figure one can also consider that the longitudinal pressure (σ_{x1}) at the floor of the stope is about 1.8 times the transverse pressure [$\sigma_x \approx 1.8 \times (\sigma_y \approx \sigma_z)$]. This observation is not true for the pressures measured at the plug/fill interface (see Fig. 8). Consequently, the transverse and vertical pressures can be evaluated using the following relationship:

$$\sigma_{y,z}(h) = \frac{0.185 \cdot \gamma H_m (H_m - z)}{B + L} \left[1 - \exp\left(-\frac{2(h-z)}{B}\right) \right] \tag{4}$$

Figure 18 shows the measured transverse pressures at the elevations 0.6 (σ_{y1}) and 7.6 m (σ_{y2}) compared to the predicted values using Equation 4. It is noted that the model reasonably well predicts the transverse pressure at the floor of the stope (σ_{y1}), but less accurately at the plug/residual fill interface (σ_{y2}).

Figure 19 shows the measured vertical pressures at the elevations 0.6 m (σ_{z1}) and 7.6 m (σ_{z2}) compared to the predicted values using Equation 4. Note again that the model predicts the vertical pressure at the floor of the stope (σ_{z1}) rather well, but predicts that at the plug/residual fill interface (σ_{z2}) less well. Other approaches developed to model the stresses in backfilled stopes have been presented in recent companion papers (Aubertin et al. 2003; Li et al. 2003, 2004).

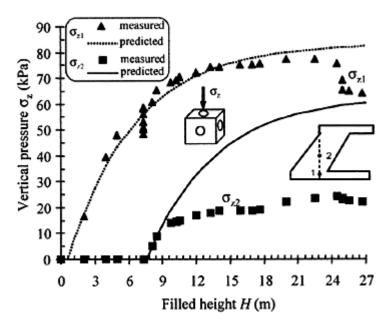


Figure 19. Comparison between experimental data and predicted curves of the vertical pressure σ_z at two elevation points (*z*1=0.6 m and *z*2=7.6 m) using Equation 4: γ = 18 kN/m3, $H_{\rm m}$ =29 m, *B*=12m, *L*=21 m.

6.3.2 Filled height and time-dependant 3D model to predict the internal pressures of the pastefill

The formulation of Equations 3 and 4 does not take into account the elapsed time during the stope filling with pastefill. In these relationships the only parameter which can vary with time is the bulk unit weight (γ) of the pastefill. However, this parameter is constant in the initial formulation of Equations 3 and 4. To take the time factor into account in these equations we propose a relationship describing the evolution of the bulk unit weight of the pastefill with time (γ^*) as follows:

$$\gamma^* = \frac{\gamma}{1 + \left(\frac{\gamma - \gamma_d}{\gamma_d}\right) \times \frac{t}{t_{\max}}}$$
(5)

where γ is initial bulk unit weight of the pastefill (kN/m³); γ_d is the dry unit weight of the pastefill (kN/m³); *t* is the time elapsed since the beginning of pastefill placement in the stope (day); t_{max} is the maximum elapsed time (day) at which $\gamma = \gamma_d$ (t_{max} is estimated to be approximately 2 years or 758 days).

Substituting Equation 5 into Equations 3 and 4 leads to the models to predict the evolution of the internal pressures of the pastefill as a function of elapsed time since the beginning of the filling as follows:

$$\sigma_{x}(t) = \frac{\gamma H_{m}(H_{m}-z)}{3(B+L)\left(1+\left(\frac{\gamma-\gamma_{d}}{\gamma_{d}}\right)\frac{t}{t_{max}}\right)}\left[1-\exp\left(-\frac{2(h-z)}{B}\right)\right]$$
(6)

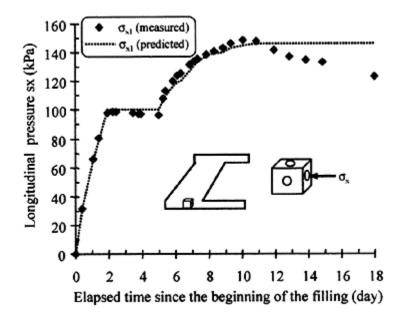


Figure 20. Comparison between experimental data and predicted curve of the longitudinal pressure σ_x at the elevation point *z*1=0.6 m using Equation 6: γ =18 kN/m3, γ_d =12.6 kN/m³, H_m =29 m, *B*=12 m, *L*=21 m, t_{max} = 758 days.

$$\sigma_{y,z}(t) = \frac{0.185 \cdot \gamma H_m (H_m - z)}{\left(B + L\right) \left(1 + \left(\frac{\gamma - \gamma_d}{\gamma_d}\right) \frac{t}{t_{\max}}\right)} \left[1 - \exp\left(-\frac{2(h-z)}{B}\right)\right]$$
(7)

Figure 20 shows the longitudinal pressure measured at the elevation 0.6 m (σ_{x1}) compared to the predicted values using Equation 6. It can be observed that the model predicts the longitudinal pressure at the floor of the filled stope (σ_{x1}) as a function of the filled height and elapsed time rather well, thus indicating that Equation 5 is well formulated.

6.3.3 3D model to predict pressure on barricades

Because of the complexity of the paste backfill the adaptation of the Rankine theory of earth pressures is not conducive to the prediction of the lateral (or longitudinal) pressure exerted on the barricade. Even though the Rankine passive and active earth pressures equations are simple to use, they nevertheless need the intrinsic parameters of the pastefill (*c* and Φ) which can be obtained from laboratory tests. From the analysis of the experimental results presented in this paper we propose a simple 3D exponential model to predict the lateral pressure exerted by the pastefill on the barricade (σ_b) as a function of filled height which is given by the following relationship:

$$\sigma_b(h) = \frac{\gamma(h-z')}{2} \exp\left(-\frac{4}{9} \cdot \frac{(B+L)(h-z')}{L \cdot B}\right)$$
(8)

where γ is the bulk unit weight of the pastefill (kN/m³); *h* is the filled height (m); *z'* is the elevation of the

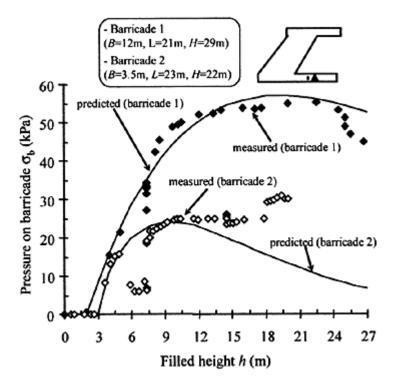


Figure 21. Comparison between experimental data and predicted curves of the longitudinal pressure σ_b on the barricade (z'1=2.1m and z'2=1.8 m) using Equation 8: $\gamma=18$ kN/m3.

point of measurement (m) in the draw point; B is the stope width (m); and L is the stope length (m).

Substituting Equation 5 into Equation 8 leads to a model to predict the lateral pressure on the barricade as a function of elapsed time since the beginning of the filling:

$$\sigma_{b}(t) = \frac{\gamma(h-z')}{2\left(1 + \left(\frac{\gamma - \gamma_{d}}{\gamma_{d}}\right)\frac{t}{t_{\max}}\right)} \exp\left(-\frac{4}{9} \cdot \frac{(B+L)(h-z')}{L \cdot B}\right)$$
(9)

Figure 21 shows the measured lateral pressure on barricades of two stopes of different size (large and small) compared to the predicted values using Equation 8. Barricade 1 (large stope) is that of the stope studied in this paper while the barricade 2 is that of another instrumented filled stope (small stope) which is not presented herein. It can be noted that Equation 8 predicts the lateral pressure on the barricade of the large stope (barricade 1) rather well, but predicts the lateral pressure on the barricade of the small stope (barricade 2) less accurately.

7 CONCLUSION

The objective of this study was to follow the evolution of internal stresses in cemented fill during and after stope filling. To reach this objective, a stope at the Doyon Gold Mine (Cambior Inc., Canada) was instrumented at various points using earth pressure cells (model TPC). The stressed induced in the pastefill ($\sigma_{-longitudinal}=\sigma_x$, $\sigma_{transversal}=\sigma_y$ and $\sigma_{vertical}=\sigma_z$) as well as on the lower wall and on the barricade (lateral or longitudinal pressure) were recorded during and after placement of pastefill. The resulting data indicate that the internal longitudinal pressure of the pastefill is higher than the transversal and vertical pressures. This tends to confirm the existence of an arching effect which develops in the stope during filling. In order to have tools for stability analysis of the filled stopes with pastefill, four 3D models were proposed to predict the internal pressures and the pressure on barricades and both as a function of the filled height (Eqs. 3 & 8) and as a function of elapsed time since the beginning of backfilling (Eqs. 5 & 9). The proposed models responses are in good agreement with the experimental data. This result is very encouraging to the on-going study of ground control using pastefill in underground mines.

The results presented herein are based on the measurements made in a specific stope filled with a specific type of pastefill and may not be applicable elsewhere.

ACKNOWLEDGMENTS

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Mining and support of tunnels in minefill at BHP Billiton Cannington Mine

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ABSTRACT: An experimental drive tunnel was mined into a backfilled stope at BHP Billiton Cannington Mine. The purpose of the drive was to investigate the effects of various blast patterns, charge weights and support systems on the stability of excavations in minefill. The results of the program showed that the addition of uncharged perimeter holes greatly reduced the amount of blast damage to the excavation surface. The addition of close-spaced perimeter drilling to the blast pattern reduced support costs and improved the stability of the tunnel under both static and dynamic loading conditions.

1 BACKGROUND

The BHP Billiton Cannington Mine is located in North-western Queensland, approximately 240 km from the town of Mt Isa. The mine location is shown in Figure 1.

The Cannington Mine produces approximately 2.4 MT per annum of silver-lead-zinc ore from its underground operations. The ore is mined by sub-level open stoping (SLOS). Stopes are mined in a combination of primary—secondary and pillarless retreat sequences on multiple mining fronts. All of the stope voids are filled after completion.

The main fill type used is paste fill. Paste fill is manufactured on surface by the addition of between 3.5% and 8% Ordinaty Portland Cement (OPC) binder to de-watered tailings from the process plant. Typical fill strengths are 600–800 kPa at 28 days. Run-of-mine mullock is sometimes added to the fill stream to reduce cement costs (Luke and Rankine, 2003). If there is no requirement for the fill to be exposed at a later date, then uncemented waste rock may be placed in the stope as fill.

In order to meet the requirements for production and blending, the mine operates multiple stoping fronts. The geometry of the orebody, the sequence and the development layout means that from time to time it is necessary to mine through filled stopes in order to access the remaining ore.

Fill mining has been practiced at Cannington since early in its production history. Results were mixed, with overbreak, and high support costs often resulting from blasting of the fill. A filled primary stope was selected for a mining trial of drill and blast mining and support practices in



Figure 1. Location.

paste and rock/paste fill. The stope was filled with a simultaneously placed combination of paste fill, and rock/paste fill as shown in Figure 2.

The outer part of the fill mass consists of paste, with only a small percentage of rock. The "rock cone" shown in the centre of the diagram consists of rock fill with interstitial paste. The properties of the fill are summarised in Table 1.

The objective of the trial was to improve the mining and support methods applied to mine fill.

Mining was carried out using a conventional twin-boom development jumbo for blast hole drilling and support installation. Fibrecrete, where used, was sprayed using a conventional wet-fibrecrete spray rig.

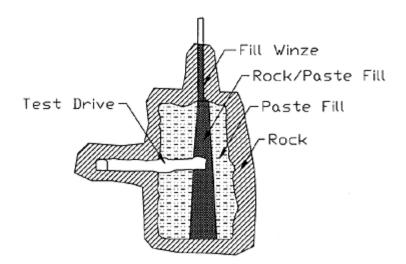


Figure 2. Distribution of fill types, stope height is 50 m.

Table 1. Summary of fill physical properties. (Luke & Rankine, 2003).

	UCS (kPa)	E (MPa)	ε _f (%)	ho (kg/m ³)	V _p (m/s)
Paste	785	14.5	2.8	2280	1550
Rock/paste	810	95.3	1.7	2650	1860

2 OBJECTIVES AND WORK PROGRAM

2.1 Program objectives

The test work was to investigate:

- The effects of explosives on fill stability.
- Methods for reducing the adverse effects of overbreak and fracturing caused by blasting.
- Improved methods of ground support.

The project was part of a larger study into the engineering properties of Cannington fill.

2.2 Work program

Five development rounds were fired. The design dimensions of the drive were 4.5 m wide \times 4.7 m high.

For each round, overbreak blast-damage patterns and sidewall fracture patterns were mapped and stereo-photographed. The blast configuration and support design was modified according to the results of each cut.

A limited program of support testing was carried out. This involved:

- Pull testing of rock bolts installed into the fill to establish the static load transfer capacity of the installed support elements.
- Observation of the dynamic loading effects from the firing of adjacent stopes.

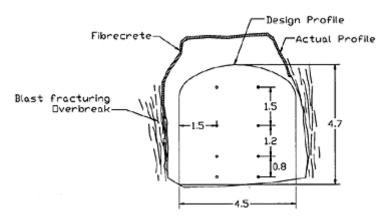


Figure 3. Round 1—Blast overbreak and fracture patterns conventional blast configuration. PF=0.18 kg/T REE ANFO.

Round	PF (REE) (kg/T)	Perimeter hole spacing (m)	Area (%)	Vol. (%)
1	0.18	N/A*	140	187
2	0.18	0.25	107	161
3	0.15	0.40	109	98
4**	0.09 (D)	0.60	104	97
5**	0.05 (D)	0.60	65	27

Table 2. Summary of blast configuration.

* N/A—Not Applicable.

** (D)—Decoupled charge.

3 PROGRAM RESULTS

3.1

Drill and blast

In order to establish a benchmark for comparison the first development round was fired using the standard blast pattern as was in practice at the time. This was an eight hole vee cut charged with packaged emulsion and a partial column of blow loaded ANFO. A vee cut was used due to difficulties with hole collapse encountered in drilling a conventional burn. The resultant excavation was examined to determine the degree of blast damage to the fill surrounding the designed perimeter. Substantial overbreak was noted, along with the formation of concentric blast fractures in the drive walls. The drill pattern, and resulting overbreak are shown in Figure 3.

Subsequent rounds were modified by the addition of perimeter control holes, changes to the explosive type, powder factor, and blast hole layout. These are summarised in Table 2.

All holes were single primed with a detonator and packaged emulsion cartridge. Rounds 1–3 were charged with partial columns of blow-loaded ANFO. Rounds 3 and 4 were charged with decoupled packaged explosives. Powder factors (PF) have been calculated to Relative Effective Energy (REE) In-HoleBulk-Strength, relative to blow loaded ANFO of



Figure 4. Blast overbreak and fracture patterns—conventional blast configuration. Powder factor=0.18 kg/T REE ANFO.

Specific Gravity 0.95, (Orica, 2003). Overbreak is measured as percentage of drive cross sectional area and volume relative to design.

The most obvious effect of the blasting was the formation of blast fractures in the drive sidewalls. Very little radial fracturing was observed indicating ductile deformation rather than brittle fracturing of the fill. Blast fractures form concentrically, parallel to the line of the blast holes (Figure 4). Convergence of the blast holes at the back of the cut results in a fracture pattern that diverges outward slightly toward the free face at an angle of 5° – 15° from the drive axis.

The resulting overbreak was substantial and the damage zone extended a considerable distance into the drive sidewalls and face. The damage radius was estimated to extend to at least 2.5 m from the blast hole. The effect of the blast damage was to seriously reduce the strength and stability of the sidewalls, to the extent that a 75 mm floor-to-floor fibrecrete layer with a 1.2×1.5 m pattern of 2.4 m friction bolts was installed maintain stability of the excavation surface.

3.2 Perimeter control

It was considered that the blast fracturing could be reduced by the addition of a series of closely spaced perimeter holes. These holes would be left uncharged, and would serve to weaken the boundary at the edge of the designed cut, reducing propagation of the shock wave and infiltration of blast gasses into the sidewalls. Round 2 used the same blast pattern and charge loading as Round 1, with the inclusion of a series of 89 mm perimeter holes drilled parallel to the drive wall. The hole centres were spaced at 0.25 m intervals as shown in Figure 5.

The degree of disturbance to the drive perimeter was greatly reduced by the addition of the perimeter holes, to the extent that "half barrels" remained in the sidewalls as shown in Figure 6. Very few blast fractures extended beyond the line of perimeter holes, indicating that the holes were effective in arresting the blast energy and diverting the stresses away from the drive walls. This conclusion was supported by the existence of deep blast damage into the face and walls beyond the ends of the perimeter holes.

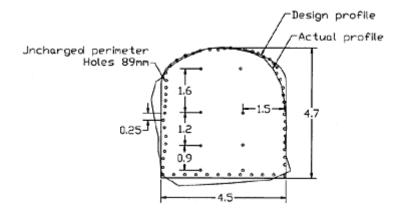


Figure 5. Blast overbreak and fracture patterns—with perimeter holes. Powder factor=0.18 kg/T REE ANFO.

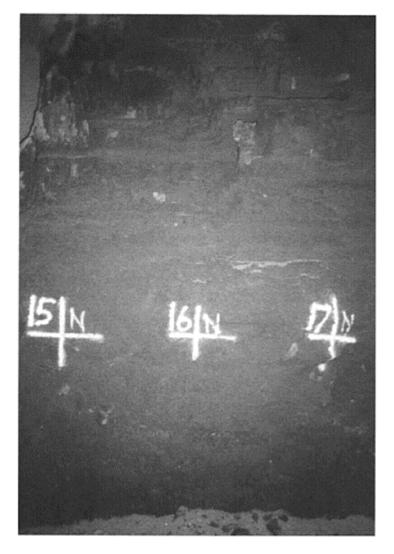


Figure 6. Sidewall half barrels—with perimeter holes cut. Powder factor=0.18 kg/T REE ANFO.

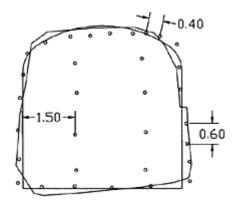


Figure 7. Blast overbreak. Round 4 with perimeter holes. Powder factor=0.09 kg/T REE ANFO.

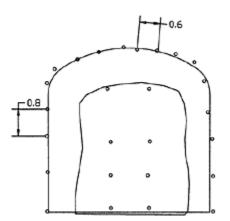


Figure 8. Blast overbreak. Round 5 with perimeter holes. Powder factor=0.05 kg/T REE ANFO.

The improved strength and cohesion of the undamaged fill reduced the requirement for ground support. Support was reduced to 50 mm of fibrecrete on the backs and shoulders of the drive, with pattern bolting. Surface support was not required for the side walls.

3.3

Powder factor

Having established the effectiveness of perimeter holes in reducing the extent of blast damage, subsequent rounds were modified with progressively lower powder factors and wider spaced perimeter holes as shown in Figures 7 and 8. The resulting area—overbreak as a function of powder factor is shown in Figure 9.

Perimeter hole spacings up to 800 mm were found to be effective in controlling overbreak, with the best results achieved for spacings of 600 mm or less. The suggested optimal blast design is shown in Figure 10.

The presence of horizontal layers in the fill reduced the benefits of a fully arched profile, due to delamination of the fill layers in the shoulders of the drive. For this reason, the design arch is somewhat flattened in the final design.

3.4 Ground support

The stability of the drive surface is greatly affected by the degree of disturbance from blasting. Blast damaged

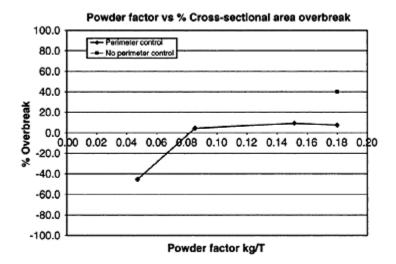


Figure 9. Cross sectional area overbreak vs Powder factor.

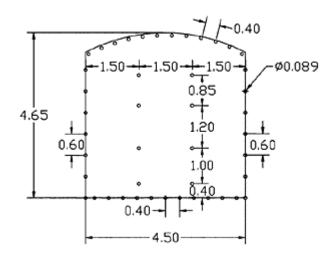


Figure 10. Optimal design—with perimeter holes. Powder factor=0.12 kg/T REE ANFO.

fill looses much of its strength due to damage to the cement bonds in the fill, and as a result of cohesion loss from disruption of the continuity of the fill itself. As with many fine-grained materials, internal pore water contributes to the cohesive strength of paste fill. It is notable that the undamaged fill maintained seepage at the drive surface, and that the fill remained somewhat plastic even after several months of exposure, whereas fractured fill dried out rapidly and deteriorated with time.

Overhead surface support is recommended. This can be either in the form of fibrecrete, or mesh (Figure 11), although fibrecrete is preferred as it restricts the amount of movement at the surface. Sidewall surface support is required if blast fracturing has occurred. Fibrecrete should be bolted using appropriate length bolts for the size of the excavation.

In the case of the test drive the bolts used were 2.4 m long, 44 mm galvanized friction bolts. These were driven into the fill without the use of a pilot hole. Pull test results for bolts installed in paste fill and rock/paste fill are shown in Figures 12 and 13.



Figure 11. Mesh support.

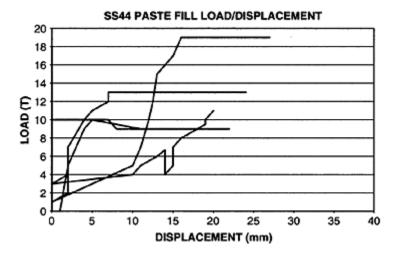


Figure 12. Load/Displacement—2.4 m friction bolt in paste fill.

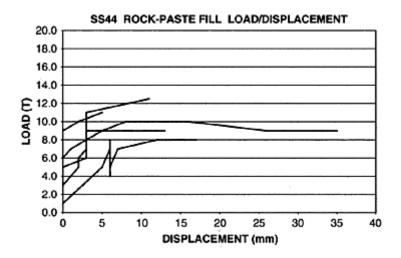
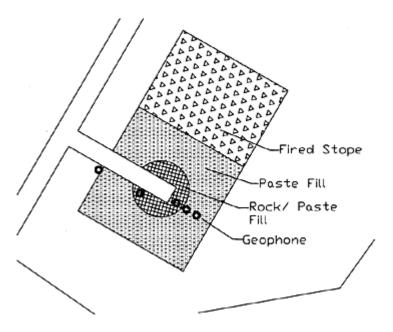


Figure 13. Load/Displacement—2.4 m friction bolt in rock/paste fill.

3.5 Dynamic loading

Subsequent to mining, the drive was subjected to repeated dynamic loading from adjacent stope firings. The blast waveforms were recorded using an array of triaxial geophones installed in the fill (Figure 14).



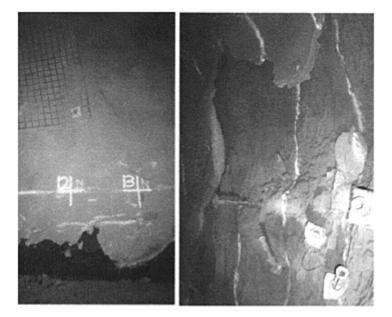


Figure 14. Geophone locations.

Figure 15. Effect of dynamic loading on blast damaged fill. Round 1: Before (left), and after (right), PPV=246 mm/s

The maximum Peak Particle Velocity (PPV) recorded was 246 mm/s perpendicular to the sidewall. The effects of dynamic loading on sections of the drive mined with, and without, perimeter holes were compared. The shock wave from the blast caused extensive damage to the sidewall and ground support of Round 1. The fibrecrete layer, and approximately 0.5 m of blast damaged fill was ejected off the wall, shown before and after in Figure 15. Round 2, which was mined with close spaced perimeter holes (0.25 m separation), was virtually unaffected by the blast (Figure 16).

Rounds 2–5, which were mined with perimeter holes, showed far greater resistance to the effects of dynamic loading when compared to Round 1, where no perimeter control was used.

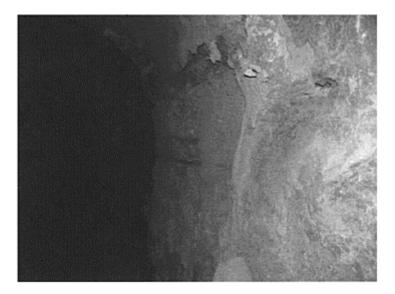


Figure 16. Effect of dynamic loading on undamaged fill. Round 2. PPV=246 mm/s. Note half barrels.

4 CONCLUSIONS

The installation of perimeter control drilling is highly beneficial to the stability of drives excavated in paste fill and rock/paste fill. The additional cost of drilling is offset by reduced overbreak, and lower support costs. Perimeter control drilling greatly reduces the adverse effects of blast damage and affords greater stability under both static and dynamic loading conditions.

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An overview on the use of paste backfill technology as a ground support method in cutand-fill mines

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ABSTRACT: This is a general overview on the use of paste backfill for ground support in underground mining operations and as such, concerns backfill design parameters (internal pressure development, required strength and mix optimisation), its pumping qualities (consistency and rheological) and delivery to an underground operation through pipelines. Emphasis was placed on the optimization of the paste backfill mix for backfill design, work safety and expense to the mining operation. This is because of the 20% representative costs related to backfilling, 15% represents the binder costs.

1 INTRODUCTION

The use of cemented paste backfill (CPB) is an increasingly important component of underground mining operations and is becoming a standard practice for use in many cutand-fill mines around the world (Landriault et al. 1997, Naylor et al. 1997). Backfill material is placed into previously mined stopes to provide a stable platform for the miners to work on and ground support for the walls of the adjacent adits as mining progresses by reducing the amount of open space which could potentially be fill by a collapse of the surrounding pillars (Barret et al. 1978). The use of underground paste backfill provides ground support to the pillars and walls, but also helps prevent caving and roof falls, and enhances pillar recovery, which enhances productivity (Coates 1981). Thus, the CPB placement provides an extremely flexible system for coping with changes in geometry of the orebody, that result in changing stope width, dip, and length (Wayment 1978). The method of the fill delivery depends upon the amount of energy required to deliver the backfill material underground which depends on its distribution cone (Arioglu 1983). The CPB is usually transported underground through reticulated pipelines.

Paste backfill is composed of mill tailings generated during mineral processing which are mixed with additives such as Portland cement, lime, pulverized fly ash, and smelter slag. The purpose of the binding agents is to develop cohesion within CPB so that exposed fill faces will be self-supporting when adjacent stopes are extracted (Mitchell 1989). With the current low metal prices, the survival of many mines depends on their ability to maximize productivity while minimizing costs. At underground cut-and-fill operation, the costs associated with backfilling must be looked at critically so that potential cost savings can be identified (Stone 1993). Backfilling is expensive in some ways, but indispensable for most underground mines to provide ground support for mine safety and mining operations. Therefore, the fill should be cost effective and capable of achieving the desired ground support and stability.

Analysis of the fill stability must consider the geometric boundaries of the fill for the best economic use of CPB. Mine openings and exposed fill faces in large underground mines vary in shape from high and narrow to low and wide. Additionally, wall rock next to the backfill may be either steeply dipping or relatively flat-lying (Mitchell 1989). The stoping sequence can be modified to reduce the number of CPB-filled stopes, or the stope geometries could be revised to reduce the strength required of CPB exposures (Stone 1993).

This paper is an overview of the use of CPB for underground ground support in mining operations, from preparation to placement underground. The paper will first briefly introduce the notion of arching effects and their importance in stability analysis of filled stopes. This will be followed by presenting the design of the required fill strength from reviews of various current design methods. The paper will discuss the optimization of CPB-mix designs (as a means to reduce costs and improve fill strength) followed by a discussion on the rheological properties of CPB. Finally, the paper will discuss CPB delivery systems and underground placement of CPB.

2 DESIGN OF THE HORIZONTAL PRESSURE ON THE FILLED STOPE SIDEWALLS

In general, self-support stresses govern backfill design and the traditional design has been that of a free standing wall, requiring a uniaxial compressive strength (UCS) equal to the overburden stress at the bottom of the filled stope. However, in many cases, the adjacent rock walls can actually help support the fill through boundary shear and arching effects. Therefore, the backfill and rock walls can be mutually supporting (Mitchell 1989). In backfilled stopes, when arching occurs (which is the case in many mines) the vertical pressure at the bottom of filled stope, an analogy similar to a trap door, is less than the weight of overlying fill (overburden weight) due to horizontal pressure transfer (Martson 1930, Terzaghi 1943). This pressure transfer is due to frictional and/or cohesive interaction between the fill and wall rock. When the pillars or stope walls begin to deform into the filled opening the fill mass will provide lateral passive resistance. Passive resistance is defined as the state of maximum resistance mobilized when force pushes against a fill mass and the mass exerts resistance to the force (Hunt 1986).

The magnitude of pressure transferred horizontally to the sidewalls can be included into the design for the required fill strength. Horizontal pressures affected by the fill arching can be evaluated by four analytical solutions which account for the existence of cohesion at the fill-sidewall interface and/or the frictional sliding along the sidewalls. These solutions are the Martson's model and its modified version, the Terzaghi's model and a proposed 3D model.

2.1 Martson's cohesionless model

Martson (1930) developed a two-dimensional arch solution to predict the horizontal pressure (σ_h) along the sidewalls of the pillars as follows:

$$\sigma_{h} = \frac{\gamma B}{2\mu'} \left[1 - \exp\left(-\frac{2K_{a}\mu' H}{B}\right) \right]$$
(1)

$$\sigma_v = \sigma_h / K_a$$

$$K_a = \tan^2 \left(45^\circ - \frac{\phi}{2} \right) \tag{3}$$

(2)

where γ =fill bulk unit weight (kN/m³); *B*=width of stope (m); *H*=total height of filled stope (m); μ' = tan δ' =coefficient of sliding friction between fill and sidewalls; δ' (degree)=angle of wall friction (may be assumed between $1/3\phi$ to $2/3\phi$); ϕ =angle of internal friction of fill (degree); σ_v =vertical pressure at the floor of the stope (kPa); K_a =coefficient of active earth pressure (see Eq. 3).

2.2 Modified Martson's cohesionless model

Aubertin et al. (2003) proposed a modified version of the Martson's two-dimensional arch solution which was originally defined using active earth pressure (K_a) and wall sliding friction. The modified version for predicting the horizontal pressure (σ_{hH}), at a depth *H*, along the sidewalls of the pillars is given as follows:

$$\sigma_{hH} = \frac{\gamma B}{2 \tan \phi'_f} \left[1 - \exp\left(-\frac{2KH \tan \phi'_f}{B}\right) \right]$$
(4)
$$\sigma_{vH} = \sigma_{hH}/K$$
(5)

where γ =fill bulk unit weight (kN/m³); *B*=width of stope (m); *H*=total height of filled stope (m); Φ **f** = fill effective angle of internal friction (degree); (σ_{vH} = vertical pressure at the floor of the stope (kPa); *K*=coefficient of fill pressure. *K* will correspond to three different states (K_a , K_0 , K_p) given by the following relationships:

$$\begin{cases}
K = K_0 = 1 - \sin \phi'_f \\
K = K_a = \tan^2 (45^\circ - \phi'_f / 2) \\
K = K_p = \tan^2 (45^\circ + \phi'_f / 2)
\end{cases}$$
(6)

where K_0 =coefficient of fill pressure at rest or in place (0.4 to 0.6); K_a =coefficient of active fill pressure (0.17 to 1); K_p =coefficient of passive fill pressure (1 to 10).

However, in a filled stope the active fill pressure condition (K_a) seems improbable. In Equation 6, the coefficient of fill at rest pressure can alternatively be evaluated using this well known relationship as follows:

$$K_{0} = \frac{\nu}{1 - \nu} \tag{7}$$

where *v*=Poisson's ratio of the fill $(0.3 \le v \le 0.4)$.

2.3 Terzaghi's cohesive model

Terzaghi (1943) also developed a two-dimensional arch theory for predicting the horizontal pressure (σ_h) along the pillar walls and this is given by:

$$\sigma_{h} = \frac{(\gamma B - 2c)}{2\tan\phi} \left[1 - \exp\left(-\frac{2KH\tan\phi}{B}\right) \right]$$
(8)

 $\sigma_v = \sigma_h / K$

$$K = 1/\left[1 + 2\tan^2\left(\phi\right)\right] \tag{10}$$

(9)

where γ =fill bulk unit weight (kN/m³); *c*=cohesive strength of fill (kPa); *B*=width of stope (m); *H*=depth below fill toe (m); *tan* Φ =coefficient of internal friction of fill; Φ =angle of internal friction of fill (degree); *K*=coefficient of fill pressure (see Eq. 10).

2.4 Proposed 3D model

Belem et al. (2004) proposed a three-dimensional model (companion paper) which implicitly takes into account the arching effects to predict the horizontal pressures, both the longitudinal pressure (σ_x) and the transverse pressures (σ_y). The model is given as follows:

$$\sigma_{x} = \frac{\gamma H_{m}(H_{m} - z)}{3(B+L)} \left[1 - \exp\left(-\frac{2(h-z)}{B}\right) \right]$$
(11)

$$\sigma_{y} = \frac{0.185 \cdot \gamma H_{m} (H_{m} - z)}{B + L} \left[1 - \exp\left(-\frac{2(h - z)}{B}\right) \right]$$
(12)
$$(\sigma_{z} = \sigma_{y}) = \sigma_{y}$$

where γ =bulk unit weight of the fill (kN/m³); $H_{\rm m}$ =total height of filled stope (m); z=elevation point of measurement (m): z=0 at the floor of the stope, $z=H_{\rm m}$ at the fill toe ($z\leq h\leq H_{\rm m}$); B=width of stope; L=strike length of stope (m).

3 DESIGN FOR CPB REQUIRED STRENGTH

The required strength for paste backfill depends upon its intended function. For a ground support role, the required uniaxial or unconfined compressive strength (UCS) of the fill should be at least 5 MPa whereas, for monly lower than 1 MPa (Stone 1993). Previous workfree-standing fill applications, the UCS can be comindicates that the UCS of the fill mass can range to between 0.2 MPa and 5 MPa, while the UCS of the surrounding rock mass is between 5 MPa and 240 MPa.

3.1 Vertical support of backfill

The mechanical effects of fill are different from those of primary ore pillars. Research and *in situ* testing have shown that fill is incapable of supporting the

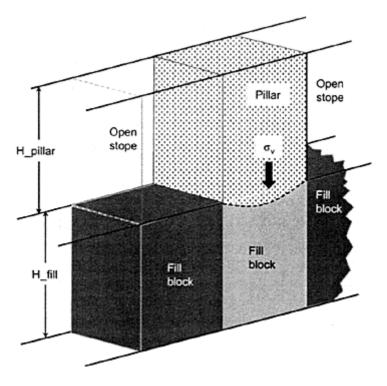


Figure 1. Schematic representation of vertical loading of a pillar into the fill mass.

total weight of overburden (σ_v) and acts only as a secondary support system (Cai 1983). The fill rigidity can range from 0.1 GPa to 1.2 GPa while the surrounding rock mass rigidity varies from 20 GPa to 100 GPa. As discussed by Donavan (1999), it is possible to

assume that any vertical loading will be a result of roof deformation (Fig. 1) and that the design UCS can be estimated by the following relationships:

$$UCS_{design} = \left(E_{p} \frac{\Delta H_{p}}{H_{p}}\right) FS$$
⁽¹⁴⁾

(15)

(16)

where E_p =rock mass or pillar elastic modulus (kPa); ΔH_p =strata length variation (m); *FS*=factor of safety.

When the stope walls deform before backfilling, the maximum load will probably never approach the total weight of the deformed overlying strata (Donavan 1999) and the design UCS can be estimated by following relationships:

$$UCS_{design} = k(\gamma_p H_p)FS$$

where *k*=scaling constant which must vary from 0.25 to 0.5; γ_p =strata unit weight (kN/m³); H_p =strata height below surface (m); *FS*=factor of safety.

Numeric modeling can also be used to determine the required stiffness or strength of a CPB to prevent subsidence due to the roof deformation. The results can be very useful in indicating the amount of the paste backfill desired. Modeling can be done with either of the FLAC (2D and 3D) codes. Physical modeling, such as with a centrifuge, also can offer an alternative to numeric modeling, but its application is usually limited to simple gravitational models without high tectonic or *in situ* horizontal stresses (Stone 1993).

3.2 Development through backflll mass

When one wants to cut an access gallery to a new ore-body through the paste backfill (Fig. 2), it is necessary to consider the original design criteria. This design considers a fill mass to be more than two contiguously exposed faces after blasting adjacent pillars or stopes. As a result, the walls confining the fill are removed and the fill block is subjected to gravity loading similar to a uniaxial compression sample (Yu 1992). The design UCS can be estimated by the following relationships:

$$UCS_{design} = (\gamma_f H_f) FS$$

where γ_f =fill bulk unit weight (kN/m³); H_f =total fill height (m); FS=factor of safety.

3.3 Pillar recovery

In order to maximize ore recovery, it is very common to return for mine pillars after primary ore recovery. While this is being done, large vertical heights of massive paste backfill may be exposed. For delayed paste backfill, as used in open stoping operations, the fill must be stable when free-standing wall faces are exposed during pillar recovery (Fig. 3). It is necessary that the fill has sufficient strength to remain free-standing during and after the process of pillar extraction by resisting the blast effects. Figure 3 illustrates a possible failure mechanism which can occur after a stope blast. Depending upon the mining schedule, high strength for such engineering materials may not be required for the short term (Hassani & Archibald 1998).

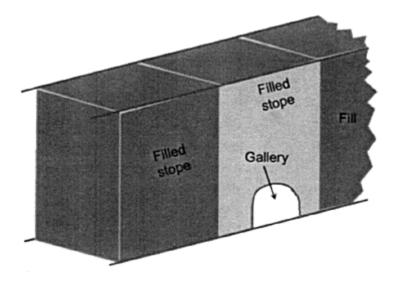


Figure 2. Digging an access gallery through the fill mass.

In the absence of numeric modeling, many mine engineers still rely on two-dimensional limit equilibrium analyses along with a calculated safety factor (FS) to determine fill exposure stability. These analyses typically result in an over-conservative estimate of the limiting strength (Stone 1993) which increase the costs of backfilling operations.

In recent years, however, 2D- and pseudo-3D empirical models have been developed to account for arching effects, cohesion and friction along sidewalls (Mitchell et al. 1982, Smith et al. 1983, Arioglu 1984, Mitchell 1989a & b, Mitchell & Roettger 1989, Chen and Jiao 1991, Yu 1992). All these design methods use the concept of a confined fill block surrounded by the wall rock.

3.3.1 More than two exposed faces

Equation (16) should be used if there are more than two contiguously exposed faces after blasting adjacent pillars or stopes (Fig. 4).

3.3.2 Narrowly exposed fill face

This design method accounts for arching effects on confined fill by adjacent stope walls (Fig. 5) using

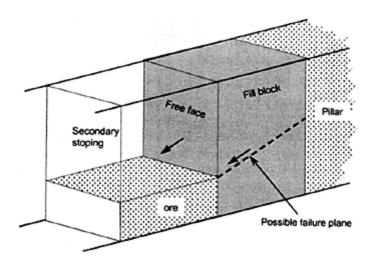


Figure 3. Fill block failure mechanism during secondary stope mining.

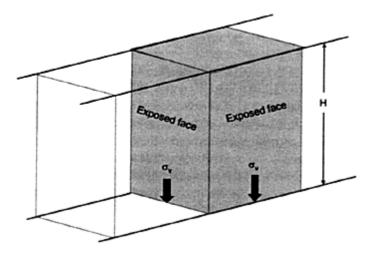


Figure 4. Schematic of a fill block with three exposed faces.

Terzaghi's vertical pressure model (Eq. 9). Based on 2D finite element modeling, Askew et al. (1978) proposed the following formula to determine the design fill compressive strength:

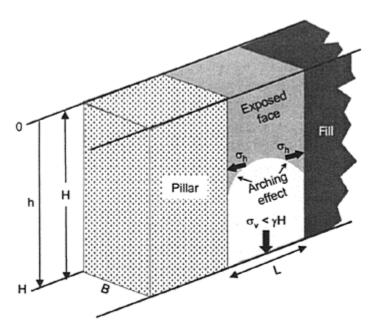
$$UCS_{design} = \frac{1.25B}{2K \tan \phi} \left(\gamma - \frac{2c}{B} \right) \left[1 - \exp\left(-\frac{2HK \tan \phi}{B} \right) \right] FS$$
(17)

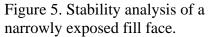
where B= width of stope; K=coefficient of fill pressure (see Eq. 10); c=cohesive strength of fill (kPa); Φ =angle of internal friction of fill (degree); γ =bulk unit weight of the fill (kN/m³); H=total height of filled stope (m); FS=factor of safety.

The fill cohesion (c) and its angle of internal friction (Φ) can be obtained from triaxial tests performed on laboratory or *in situ* backfill samples.

3.3.3 Exposed frictional fill face

This design refers to an exposed fill where both opposite sides of the fill are against stope walls (Fig. 6). By





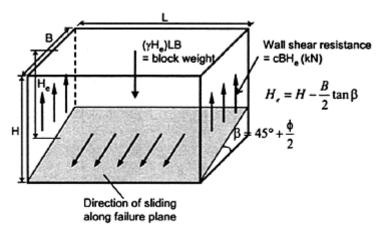


Figure 6. Confined block with shear resistance mechanism (after Mitchell et al. 1982).

assuming that there is shear resistance between the fill and stope walls due to the fill cohesion, the design UCS can be evaluated by the following relationship (Mitchell 1982):

$$UCS_{design} = \frac{\left(\gamma L - 2c\right)\left[H - \frac{B}{2}\tan\left(45^{\circ} + \frac{\phi}{2}\right)\right]\sin\left(45^{\circ} + \frac{\phi}{2}\right)}{L}FS$$
(18)

where γ = fill bulk unit weight (kN/m³); *c*=cohesive strength of fill (kPa); *L*=strike length of stope (m); *B*= width of stope (m); *H*=total height of fill (m); Φ =angle of internal friction of fill (degree); *FS*= factor of safety.

Again, the fill cohesion (c) and its angle of internal friction (Φ) can be obtained from triaxial tests performed on laboratory or *in situ* backfill samples.

3.3.4 Exposed frictionless fill face

The compressive strength of paste backfill is mainly due to the binding agents and any strength contributed from friction can be considered negligible for the long term (i.e. $\Phi = 0$). For a frictionless material (Fig. 7), cohesion is assumed to be half of the UCS (*c*= UCS/2). Thus, the design UCS can be evaluated by the following relationship proposed by Mitchell et al. (1982):

$$UCS_{design} = \frac{(\gamma L - 2c) \left(H - \frac{B}{2}\right) \sin(45^\circ)}{L} FS$$
(19)

where γ = fill bulk unit weight (kN/m³); *c*=cohesive strength of fill (kPa); *B*=width of stope (m); *L*=strike length of stope (m); *H*=total height of fill (m); *FS*=factor of safety (ca. 1.5).

In Equation 19, the fill cohesion (c) can be obtained from laboratory confined compression tests on backfill samples.

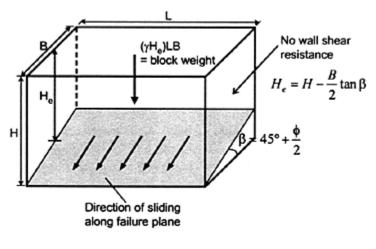


Figure 7. Confined block without shear resistance mechanism of frictionless fill (adapted from Mitchell et al. 1982).

The stability of a free standing backfill (Fig. 7) can also be determined from physical model tests. Based on centrifugal modeling tests, Mitchell (1983) proposed a formula for evaluating the design UCS which is given by:

$$UCS_{design} = \frac{\gamma L H}{L + H} FS \tag{20}$$

where γ =fill bulk unit weight (kN/m³); *L*=strike length of stope (m); *H*=total height of fill (m); *FS*=factor of safety.

3.4 Ground support

After passive resistance has been mobilized by the fill, the strength increase in the surrounding pillars will be equal to the magnitude of the passive fill pressure. So, the main stabilizing effect of the fill is to give increased lateral confinement pressure to the pillars (Fig. 8). The compressive strength of the pillar increases according to the following formula (Guang-Xu & Mao-Yuan, 1983):

$$UCS'_{p}=UCS_{p}+([\gamma H+q]K_{p-f})K_{p-p}$$

(21)

$$K_{p-f} = \tan^2 \left(45^\circ + \frac{\phi_f}{2} \right) \tag{22}$$

$$K_{p-p} = \tan^2 \left(45^\circ + \frac{\phi_p}{2} \right) \tag{23}$$

where UCS'_{p} =pillar compressive strength with fill (kPa); UCS_{p} =pillar strength before the stope filling (kPa); γ =fill bulk unit weight (kN/m³); q=surcharge loading (kPa); H=total height of fill (m);

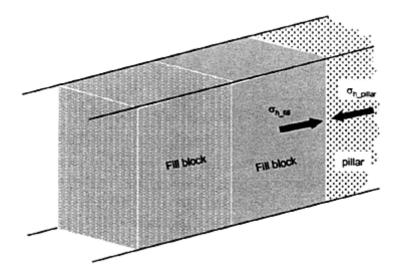


Figure 8. Schematic of pillar confinement by the fill block.

3.5 Working platform

For cyclic backfilling operations, as in cut-and-fill stoping, the fill in each operation must serve as a platform for both mining equipment and personnel and typically requires high strength development for the short term. A standard bearing capacity relationship that has been developed from civil engineering techniques for design of shallow foundations has been found to be applicable to paste backfill. The fill top surface bearing capacity, Q_f (kPa), can be determined using Terzaghi's expression (Craig 1995):

 $Q_{j}=0.4\gamma BN_{\gamma}+1.2cN_{c}$ (24)

$$N_{\gamma} = 1.8(N_q - 1)\tan\phi \tag{25}$$

$$N_c = \frac{\left(N_q - 1\right)}{\tan\phi} \tag{26}$$

$$N_q = \tan^2 \left(45^\circ + \frac{\phi}{2} \right) \exp(\pi \tan \phi)$$
 (27)

where γ =bulk unit weight of the fill (kN/m³); *c*= cohesive strength of fill (kPa); *B*=width of square footing at surface contact position (m); *N* γ =unit weight bearing capacity factor;

 $N_{\rm c}$ =cohesion bearing capacity factor; $N_{\rm q}$ =surcharge bearing capacity factor; Φ =angle of internal friction of fill (degree).

Equation 24 assumes that backfill bearing is by a square footing, which is a reasonable representation of the footprint of a mine vehicle tire (Hassani & Archibald 1998). Equation 25 was developed by Hansen (1968). For the mine vehicles (Fig. 9), the

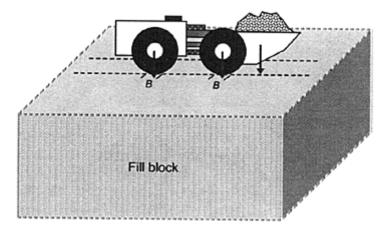
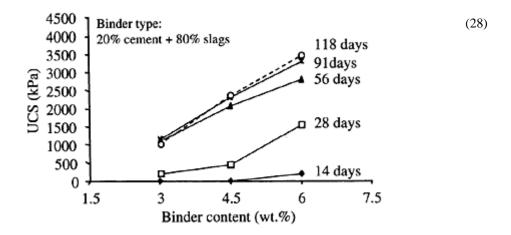


Figure 9. Schematic of working platform (adapted from Hassani & Bois 1992).

contact width, *B*, corresponds to the tire contact width and can be determined by the following relationship (Hassani & Bois 1992):



where F_t =tire loading force (kN); *p*=tire air pressure (kN/m²).

4 OPTIMIZATION OF PASTE BACKFILL MIX DESIGNS

Once the required strength has been determined, the mix variables can be optimized to provide the desired mix, which achieves the target strength with the lowest cement usage. The mix variables under consideration include the binder content and type, mill tailings grain size distribution and mineralogy, solids concentration, and the mixing-water chemistry. For the design of a certain uniaxial compressive strength (UCS_{design}), these variables can be adjusted to produce an optimal mix design (Stone 1993).

The other essential requirement is that backfill must inexpensive. Typical costs of backfill range from \$2 to \$20 per cubic meter, depending on the service required. These costs can be a significant contribution to the operating costs of the mine. Where cemented backfill is used, these costs tend to be between 10 and 20% of the total operating cost of the mine and cement represents up to 75% of that cost (Grice 1998). Optimization of CPB-mix designs can reduce binder usage and can offer significant cost savings (Fall & Benzaazoua 2003).

4.1 Laboratory optimization of CPB mix designs

Figure 10 shows the main components which can affect the quality of cemented paste backfill such as

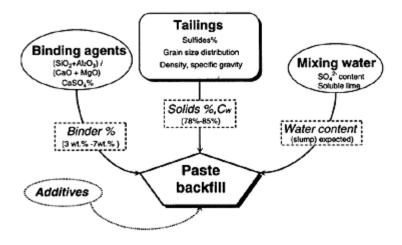


Figure 10. Schematic diagram illustrating the different components of cemented paste backfill (Benzaazoua et al. 2002).

the binding agents, mill tailings mineralogy, mill tailings grain size, the density and solids percentage of tailings and finally, the mixing-water geochemistry (Benzaazoua et al. 2002).

Each component plays an important role for the backfill transportation, its delivery and its strength development in the course of curing time. Typical binder percentages are 3 to 7% by weight of the paste fill. Numerous laboratory test results have reported that the backfill strength is a function of binder content for a given curing age (Fig. 11), but this relationship is specific to each mine (e.g. Benzaazoua et al. 1999, 2002, 2004).

4.1.1 Cement and others binders

Hardening of the fill occurs as bonds are formed between fill particles at grain contact points. Many different types of binding agents are used, but the most common is ordinary Portland cement (OPC). Admixtures with pozzolanic materials are also used to curb costs by reducing the amount of Portland cement needed for hardening. Fly ash (FA) and smelter ground blast furnace slags (BFS) are the most popular pozzolans used as admixtures. The results of cement dissolution tests performed by Benzaazoua et al. (2004) showed that in either concrete or mortar, the hardening processes within the pastefill are not only due to the cement hydration but also to the precipitation of hydrated phases from the pore water of the paste. Figure 12 illustrates that paste backfill hardening occurs in two main stages: the first stage (*dissolution-hydration*) which is dominated by the dissolution reactions and the second stage (*precipitation and hydration*) which is characterized by the precipitation reactions and direct hydration of the binder. More details on this subject can be found in Benzaazoua et al. (1999,2002,2004).

Water is necessary to ensure that proper hydration of the cement occurs. If proper hydration of the cement does not occur, the fill will not meet its required strength and stiffness. Since tailings backfill is fairly

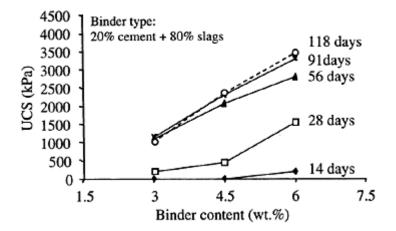
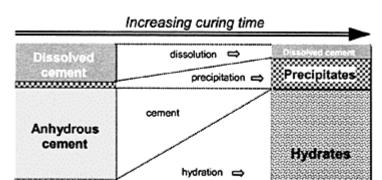


Figure 11. Example of variation of UCS as a function of binder content at different curing times of 14, 28, 56, 91 and 118 days (after Benzaazoua et al. 2003).





Phase II

Figure 12. Schematic illustration of the time-depending importance of the reactions of dissolution and precipitation of the binder in the

hardening process of the pastefill (after Benzaazoua et al. 2004).

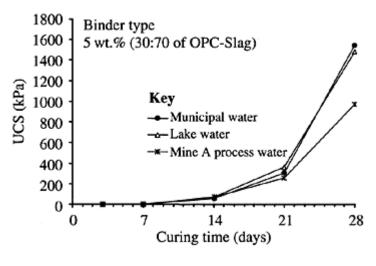


Figure 13. Effect of the mixing water on strength development within paste backfill mixtures with mine A tailings (after Benzaazoua et al. 2002).

saturated to begin with and additional water is usually required to pump it underground, the water content of tailings backfill is always in far excess of what is required for hydration of the Portland cement. The main concern then is the pH of the water and the amount of sulfate salts present in the water. Acidic water and sulfate salts can attack the cement bonds within the fill, leading to a loss of strength, durability, and stability (Benzaazoua et al. 2002, 2004).

Figure 13 shows that when using cement-slag binder with the same tailings sample mixed with three different waters, the strength development is slow for all three waters for a curing time of 14 days (Benzaazoua et al. 2002). Beyond this curing date and at a curing time of 28 days UCS reached a value of about 1600 kPa with the sulfate-free waters (municipal and lake waters) and only 1000 kPa with the sulfate-rich water (mine A process water).

4.1.2 Mixing process

Some amount of water is added to set the resultant paste backfill to attain the desired slump value. The

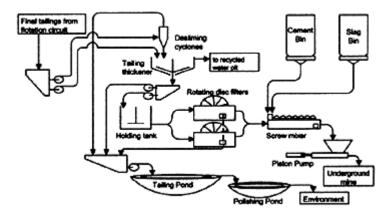


Figure 14. Paste backfill plant flow sheet at Louvicourt mine in Canada (after Cayouette 2003).

slump must vary from 12.4 to 25.4 cm (5 to 10 in) which correspond to solids concentrations of 78% to 82% by weight. Slump is a measure of the drop in height a material undergoes when it is released from a cone-shaped slip mold. Determination of slump provides a way to characterize a material's consistency that can be related to its transportability.

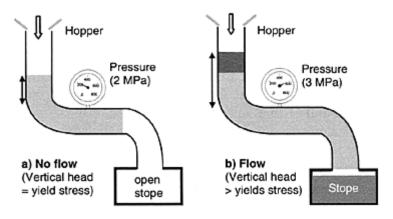
The resultant paste backfill mixtures were poured into plastic cylinders 10.25 cm in diameter and 20.5 cm height, sealed and cured in a humidity-controlled chamber at approximately 90–100% relative humidity (similar to underground mine working conditions). The pastefill samples were then subjected to uniaxial compression tests for periods of 7, 14, 28, 56 and 91 days.

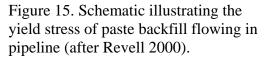
4.2 CPB preparation at the backfill plant

Figure 14 shows a typical flow chart for a backfill plant. The final mill tailings are first fed to a high-capacity thickener to increase their solids percentage to approximately 55% to 60% by weight. To aid filtration some flocculent is added. The thickened tailings are then pumped from the thickener to a high-capacity holding tank (after cyanide destruction). From the surge tank, the thickened tailings are fed by a gravity circuit to disc filters operating alone or in parallel to produce a filter cake with a solids percentage of approximately 70% to 82%. The filter cake is then discharged onto a belt (or reversible) conveyor and is then fed to a screw feeder for weighing. Filter cake batches are mixed in a spiral (or screw) mixer with cement and water is added to produce a paste with a specified slump (127 to 254 cm). The mixed paste is dropped into a surge hopper and discharged underground under vacuum (by gravity or using concrete pump).

5 RHEOLOGICAL PROPERTIES OF CPB

Paste backfill consists of the full size fraction of the tailings stream prepared as a high slurry density. The slurry behaves as a non-Newtonian fluid, which means





that it requires an applied force to commence flowing (Fig. 15).

Toothpaste is an example of a non-Newtonian fluid that is commonly used and the yield stress (applied force) explains why you have to squeeze the toothpaste out of the tube. The paste has a higher viscosity and exhibits plug flow when transported in a pipe. The outer portions of the slurry shear against the sidewall of the pipe and the central core travels as a plug (Grice 1998). The flow of paste backfill in pipeline is entirely governed by their rheological properties. Rheology is the science about flow and deformation of matter.

5.1 Rheological models of CPB

The main mode for paste backfill flow in pipelines is the full-fall. Full-pipe flow refers to the situation where the flowing paste forms a continuum and there is no air-filled gap or discontinuities (vacuum "holes") anywhere in the pipeline segment under consideration (Li et Moerman. 2002).

The most fundamental relationship in the rheology of a non-Newtonian fluid is that between the shear rate, $\dot{\gamma}(s^{-1})$ and pipe wall shear stress, $\tau_w(Pa)$. Once this relationship is known, the behaviour of the fluid in all flow situations can be deduced. All non-Newtonian fluid rheology can be derived from the most general Herschel-Bulkley model given by:

$$\tau_{w} = \tau_{0} + k \left(\frac{dV}{dr}\right)^{n} = \tau_{0} + k \dot{\gamma}^{n}$$
⁽²⁹⁾

where \mathbf{T}_0 =yield stress (Pa), *k*=consistency parameter or viscosity (Pa.s), (dV/dr)=paste angular velocity or shear rate (s⁻¹); *r*=point of velocity profile (m), *R*=radius of the pipe (m), *V*=paste linear velocity (m/s), *n*=flow parameter.

For the Newtonian fluids, $\tau_0 = n = 0$; for the pseudoplastic fluids, $\tau_0 = 0$ and n < 0; for dilatant fluids, $\tau_0 = 0$ and n > 0; for Bingham plastic fluids, $\tau_0 > 0$, n=0 and $k=\eta=$ plastic viscosity in Pa.s

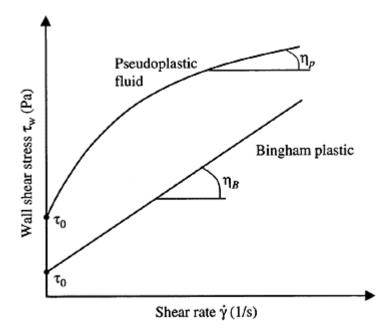


Figure 16. Rheology models for timeindependent fluids.

(Fig. 16); for yield pseudoplastic fluids, $\tau_0 > 0$ and n > 0 (Fig. 16); for yield dilatant fluids, $\tau_0 > 0$ and n > 0.

Paste backfills are non-Newtonian fluids and their rheology is approximately timeindependent during its transport in pipeline. Most paste backfill show an appreciable yield stress and are Herschel-Bulkley fluids (Eq. 29). Some paste backfills are Bingham plastic in limited shear rate ranges. Others are yield pseudoplastic or yield dilatant, with the former more common than the latter. The relationship between the pseudo shear rate, 8 V/D, and the shear stress at the pipe wall, τ_{w} is given by:

$$\tau_{w} \approx \frac{D\Delta P}{4L} = \eta \frac{8V}{D} \left[1 - \frac{4}{3} \left(\tau_{0} \frac{4L}{D\Delta P} \right) + \frac{1}{3} \left(\tau_{0} \frac{4L}{D\Delta P} \right)^{4} \right]^{-1}$$
(30)
(30)

where \mathbf{T}_{0} =yield stress (Pa), η =paste plastic viscosity (Pa.s), ΔP =differential pressure in the pipe (Pa); *D*=internal pipe diameter (m), *L*=pipe length (m); *V*=paste laminar velocity (m/s).

The effective pipes diameter (*D*) for paste backfill transport is ranged between 10 cm and 20 cm (4 and 8 in). Paste flow velocity varies from 0.1 m/s to 1 m/s. The practical pumping distance of paste can reach 1000 m longitudinally (L_h) and unlimited vertically (L_v).

5.2 Standard measurements of the CPB consistency

In practice, it is not easy to obtain the true rheological properties of pastes due to the complexity of the experimental devices. This makes difficult, even impossible, the determination or the prediction of a pastes viscosity which depends on several factors. That is why the standard slump test (used in concrete experiments) is widely used, due to its simplicity, to determine paste backfill consistency. Slump is a measure of the drop in height of a material when it is released from a truncated metal cone, open at both ends and sitting on horizontal surface (Fig. 17). Determination of the slump provides a way to characterize a material's consistency that can be related to its transportability (Clark et al. 1995). According to Landriault et al. (1997), the ideal slump of the paste must be in a range between 150 mm (6 in) and 250 mm (10 in) to facilitate the flow of cemented paste backfill by its pumping underground.

Solids concentration is often used to compare the composition of mixes, particularly in batch. Although solids percentage does not provide a direct indication of a material's consistency, in some cases it can be correlated to the slump, which does.

In order to achieve the same mix consistency from batch to batch, consistency can be measured by monitoring the electrical power used by a motor turning the paddles of a mixer. The mixer is started and water is added until the power required by the motor corresponds to the target power for the mix consistency desired (Brackebusch 1994, Landriault & Lidkea 1993). Using this arrangement requires only that slump be correlated to consistency and consistency be correlated to power. It is also possible to predict what pressure gradient a mix will produce based on power once a correlation has been established between slump and pressure loss.

5.3 A lternative methods for rheological factor measurements

To correctly define the rheology of paste backfill both the yield stress (τ_0) and the viscosity (η) need to be measured. Most current tests measure only one rheological factor.

The relationship between the factor measured and either of the two fundamental rheological parameters is not obvious. In most cases, **\tau_0** and η =cannot be calculated from the factor measured, but can only be assumed to be related. According to

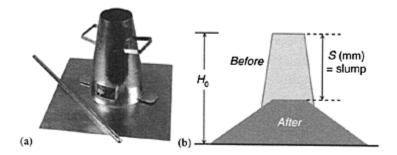


Figure 17. Paste backfill consistency measurement by slump tests: a) slump cone mold; b) schematic view of the slump test.

Ferraris (1999), slump, penetrating rod and K-slump tests are related to the yield stress (τ_0) because they measure the ability of paste to start flowing. The remolding test, LCL apparatus, vibrating testing apparatus, flow cone, turning tube viscometer, filling ability and Orimet apparatus are related to the viscosity because they measure the ability of paste to flow after the applied stress (vibration or gravity) exceeds the yield stress.

Recently, a modification of the slump cone was developed to allow the measurement of viscosity (Ferraris & de Larrard 1998). As mentioned earlier, the standard slump test can only be correlated with the yield stress (τ_0). The modification consists of measuring not only the final slump height but also the speed at which the concrete (or paste backfill) slumped. The method consists of measuring the time (*T*) for a plate resting on the top of the concrete to slide down with the concrete (or paste backfill) a distance of 100 mm (Fig. 18).

The yield stress, τ_0 , can be calculated from the final slump (S), using the following empirical equation proposed by Ferraris & de Larrard (1998):

$$\tau_0 = \frac{\rho(H-S)}{a} + b \tag{31}$$

where ρ =paste density (kg/m³); S=final slump (mm); a, b=material constants, H=300 mm is the cone height. For the concrete paste, *a*=347 and *b*=212.

From a range of paste backfill slump values (130–250 mm), the viscosity can be determined from the 100 mm slump time (T) using an empirical equation that was developed by Ferraris & de Larrard (1998):

$$\eta = k\rho T$$

(32)

where η =viscosity (Pa.s); *k*=material constant (*k*=0.025 for concrete); ρ =paste density (kg/m³); *T*=slumping time (s).

Other authors (Nguyen & Boger 1985) have suggested adapting the laboratory vane shear test for the

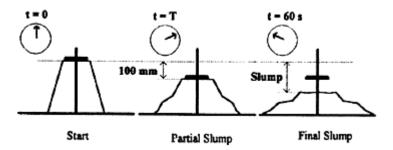


Figure 18. Schematics of the modified slump cone test based on slumping time *T* measurement (after Ferraris & de Larrard 1998).

measure of paste yield stress (τ_0). This test allows obtaining a torque-angular deformation curve of the paste whose peak corresponds to the maximum torque (Γ_m). If these these parameters are known, the yield stress can then be calculated by the following relationship:

$$\tau_0 = \frac{\Gamma_m}{\frac{\pi D^3}{2} \left(\frac{H}{D} + \frac{1}{3}\right)}$$
(33)

where τ_0 =paste yield stress (Pa), Γ_m =maximum peak torque value (N.m), D=vane diameter (cm); H=vane height (cm).

6 CPB TRANSPORT BY PIPELINES

6.1 Type of underground distribution systems

There are three possible configurations for moving fill material from a point on the surface to the underground stopes as shown on Figure 19 (Thomas 1979).

As discussed by Thomas (1979), the "gravity/pump" system (Fig. 19) has the advantage of being totally contained underground, thus causing no disruption to surface

activities. Furthermore, the ratio of the vertical to horizontal distance is usually so favourable that little or no pumping energy is required.

The "gravity" system (Fig. 19) has the advantage of by converting vertical head to horizontal pressure progressively which allows shorter and lighter pipes to be used. The pressure at the take-off points are moderate and line failures, if any, do not disrupt the main shaft or main level of operation. The circuit can be developed progressively as the mine expands.

The "pump/gravity" system (Fig. 19) has the advantage of easy installation, inspection and maintenance,

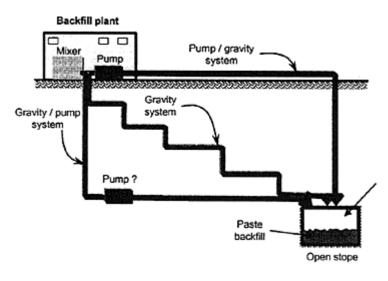


Figure 19. Basic configurations for paste backfill distribution systems (adapted from Thomas et al. 1979).

with no special underground level requirements and no disruption of the main shaft. However, such a system makes the filling operation dependent upon a pumping operation and requires a long borehole to place fill underground which results in a high pressure take-off point.

6.2 CPB transport underground

The paste backfill is delivered by pipeline to the disposal point in the stope and the friction factors generated require that high pressure pipelines be used to transport the pastefill. Pressures typically exceed 5 MPa for this type of laminar flow system. Early systems used high pressure reciprocating pumps but experience has shown that pastefill can be readily transported by gravity alone, provided that the reticulation geometry is favourable (Grice 1998).

6.2.1 Flow-loop tests of the CPB

For a given mine, a fully instrumented pipes for paste backfill flow-loop tests must be performed to determine the paste transport characteristics. Usually this is an instrumented, closed-circuit pipeline system powered by a diesel engine positivedisplacement pump. The instrumentation on the paste flow-loop tests provides essential engineering data such as flow rate (Q), friction head loss per unit length of pipe ($f=\Delta P/L$), shutdown and restart capabilities, and power consumption needed to design full-scale pipelines. Figure 20 is an example of paste flow-loop tests performed at the USBM's Spokane Research Center (Clark et al. 1995).

The calculation of the friction head loss $(\Delta P/L)$ will allows determination of the running pressures of the paste distribution system: type of volumetric displacement pump, choice of pipe diameters (D), flow rate (Q), and paste flow velocity (V). For a Bingham plastic fluid

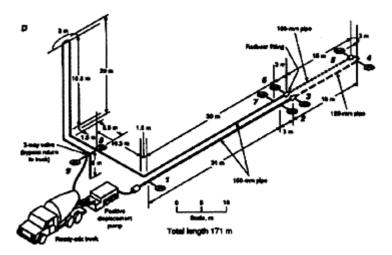


Figure 20. Pastefill flow-loop tests configuration and pressure monitoring locations (after Clark et al. 1995).

flowing in laminar regime (pastefill), the friction head loss (f) is given by the following relationship:

$$f = \frac{\Delta P}{L} = \frac{32\eta_B V}{D^2 \left(1 - \frac{4\tau_w}{3\tau_0} + \frac{1}{3} \left(\frac{\tau_w}{\tau_0}\right)^4\right)}$$
(34)

where *f*=friction head loss (Pa/m); η_B =Bingham plastic viscosity (Pa.s); τ_0 =yield stress (Pa); τ_w = wall shear stress in Pa ($\tau_w \approx D\Delta P/4L$); *D*=pipe diameter (m); ΔP =differential pressure in the pipe (Pa).

The use of rheological models such as Equation 33 requires the *a priori* knowledge of the paste Bingham plastic viscosity (η) which is very difficult to predict because it depends on several factors. That is why it is important to relate the slump value to the plastic viscosity as the relationships (Eqs. 30 & 31) proposed by Ferraris & de Larrard (1998). The pipe diameters often used vary between 100 mm (4 in) and 200 mm (8 in). For example, a paste backfill with a slump value of 180 mm (7 in) can be transported by gravity at a flow rate of 100 ton/hour in boreholes/pipes system with a 150 mm (6 in) diameter.

6.2.2 Horizontal transport distance

The horizontal transport distance (L_h) generated by a standing column of material is obtained by dividing the pressure at the bottom of the standing column (P_{bottom}) by the frictional pressure gradient or pressure loss (Clark et al. 1995). The pressure at the bottom of a standing column is obtained by taking the difference between the pressure imparted by gravity and pressure lost through frictional pressure gradient, so that horizontal transport distance (L_h) is given par the following relationship (Fig. 21):

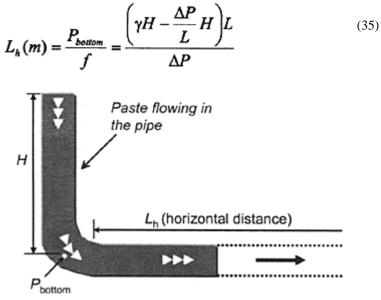


Figure 21. Schematic illustrating the calculation of the horizontal distance of paste flow.

where γ =fill bulk unit weight (kN/m³); *H*=maximum free-fall height of the paste in the paste (m); $\Delta P/L$,=friction head loss (Pa/m).

7 BACKFILL DELIVERY IN THE STOPES

Once all the transport parameters are correct, the paste backfill can be delivered to underground openings through pipelines. Figure 22 shows a general outline of a backfilled stope with its various components (fill mass, barricade, rock mass, adjacent filled stope) as well as the stress field distribution.

After the stope is backfilled with CPB its mechanical integrity can be threatened by several macroscopic factors (in opposition to the hydration process) which are going to influence the mechanical strength of the CPB and the structural stability of the filled stope. These factors which result from interactions between CPB and rock walls are, fill settlement and the drainage of its excess water, fill consolidation, stope volume, stress field distribution within the backfill mass (pressures at the floor of the stope and on the barricade), wall convergence against the fill mass, shrinkage and the arching effect.

Drainage and settlement will favour the development of a high mechanical strength of the CPB (Belem et al. 2001, 2002). On the other hand, the fill mass will be stable due to the development of arching effects depending upon the stope dimensions.

The pressures at the floor of the stope and on the barricade will have a harmful effect on the stability of the filled stope when these pressures are too high (see more details in Belem et al. in the companion paper). Consequently, it is necessary to understand these various factors which influence stope stability to ensure better ground control.

The knowledge of the magnitude of the pressures on the barricade will allow better planning of the mining sequences. The knowledge of the stress field within

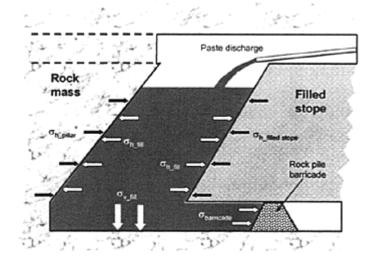


Figure 22. Schematic showing the components of a backfilled stope and the stress field distribution.

the fill mass will facilitate its stability analysis when it is considered that one of its faces may be exposed or when one wants to cut an access gallery to a new orebody through the CPB.

8 CONCLUSION

This paper is a general overview on the use of cemented paste backfill, from its design to its underground delivery. When a mining method uses paste backfill, initially one must determine the limiting strength and the pressures which will be developed in the fill according to the geometry of the opened stopes.

To meet these criteria, laboratory optimization of paste backfill mix design will be essential to determine the ideal mixture to acheive the desired limiting strength. But before beginning the stope filling, it would be necessary to know the rheological properties of the fill material. For that purpose, one will select a rheological model of paste backfill behaviour (Bingham or Pseudo-plastic) to determine the two essential parameters, yield stress and viscosity.

The pumpability of the paste backfill can be also estimated using the standard or modified slump tests. This last would allow relating the slump and the "slumping time" to the yield stress and the plastic viscosity. According to existing distribution system at the mine concerned (e.g. gravity, pumping, etc.), paste flow-loop tests are necessary to estimate the friction head loss of the pipelines for better control of the operating pressures.

With this last parameter, it would be also possible to calculate the maximum horizontal distance for the paste flow without any additional pressure. Once the paste backfill is transported underground through the pipelines to the open stopes, it will interact with the stopes and pillar walls and its initial physical and mechanical properties will evolve in the course of its curing time.

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Raise climber—supporting method for stability of raise development in Pongkor Gold Mine, Indonesia

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ABSTRACT: The Pongkor Gold Mine in Indonesia consists of three main ore bodies accessed from underground. Raising methods are used to form ore passes and waste passes. Raise boring has proved to be unsuccessfUl. Three vertical shafts have collapsed after reaming and another will require a full steel liner and concrete for 80 metres from the surface. Consequently, the raising method has been changed to Raise Climbing (Raise Climber Method). The raise climber method has been successful due to the flexibility to install the correct support wher needed. One vertical shaft, 170 metres long, has been developed using this method. The rock condition along the shaft before the excavation was known from the core drilling. The RMR method and the Q system were used to design appropriate support and reinforcement consisting generally of weld mesh and split set. Steel liner and concreting were implemented at locations where very poor rock conditions were found. By this supporting method, the lateral and vertical stress can be reduced, and the possible damage to the raise can be avoided.

1 INSTRUCTIONS

1.1 Company and access

Pongkor Gold Mine, owned by the Indonesian state company PT Antam Tbk, is located in Regency Bogor, West Java Province, 100 km South West from Jakarta, in Indonesia. It is located in very steep hilly terrain intersected by many river valleys and the mine site is at approximately 500 metres above mean sea level.

Pongkor Gold Mine is one of 6 Strategic Business Units of PT Antam Tbk (one of Indonesian State Owned Mining Company). The mining production capacity for the plant is 1,220 dmt per day and gold processing capacity is 5,000 kg per year. The processing plant can also process silver as a by-product for 30,000 kg per year. The grades of mineral content are 12.31 gpt Au and 133.63 gpt Ag (see Figure 1).

The Pongkor Gold Mine consists of three orebodies. The Ciguha orebody, which is nearly mined out, The Kubang Cicau orebody and the Ciurug orebody. In geotechnical aspects, the rock conditions vary from poor to very poor condition (RMR Method). It is caused by intensive weathered and strong alteration of the rock. Some lithologies susceptible to water degradatian The geological structures consist of joints with variations of dips also dominate the forming of quartz vein. Based on this condition, geotechnical aspect is the important thing for starting all the

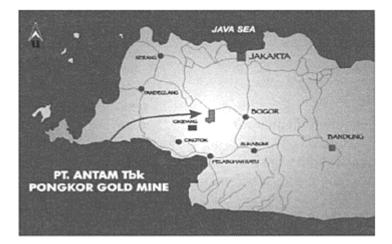


Figure 1. Pongkor Gold Mine is located close to the capital city of Jakarta.

underground activities especially for shafts (ore passes and waste passes).

The average rainfall in Pasir Saledeng (25 km South of Pongkor), is 3.5 meters per year. Although heavily rainfall can occur in every month, the period from November to April is noted to have greater rainfall. Temperatures vary between approx. 18 to 35°C.

1.2 Exploration history

Between year 1988–1992, Detailed Exploration is started at Pongkor with regional and detailed geological mapping, geochemical stream sediment and panconcentrate analyses, and also follow up by systematic core drilling. Four gold prospect were found at Pasir Jawa vein, Ciguha vein, Kubang Cicau vein and Ciurug vein.

Feasibility study done in year 1992, followed by development of adit from main portal to the veins. Finally, in April 1994 Pongkor Gold Mine started its gold production.

Later on, in year 2000, Pongkor's geologists found another reserve, called Gudang Handak vein. With production capacity per year of 5,000 kg gold and 30,000 kg Silver, Pongkor's mine life will be 20 years (1994–2014).

2 GEOLOGY

The Pongkor project area is an exposed window of hydro-thermally altered volcanic rocks that host anomalous concentrations of gold and silver mineralization in a series of quartz veins. Within this area 9 quartz vein deposits have been recognized, but to date only three veins (Ciguha Utama, Kubang Cicau and Ciurug) have been systematically explored.

The vein systems are found in outcrop at elevations between 550 and 750 metres above sea level. They are typically orientated between NW-SE and N-S with variable dips from 55 to 90 degrees and average widths of 0.8 to 24 metres.

2.1 Description

This paper generally pertains to details of the three actively mined deposits.

Mineralogy: Vein mineralogy is dominated by quartz with varying quantities of clay minerals and may contain significant quantities of manganese oxide

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Figure 2. Stratigraphic of Pongkor area.

which adversely affects mining conditions. The Ciurug Vein is shown to have very much less manganese oxide present than the Ciguha/Kubang Cicau Veins.

Structure: The region is considered to contain little or no major post-mineralisation faulting. The primary regional fracture system is generally parallel to the veins and may also exhibit discontinuities in dip and strike.

Rock strength: The quartz vein lodes are typically high to very high strength which decreases where clay minerals are present. In contrast, the host-rock mass strength is low to medium.

The Hanging and Footwall lithologies are fine to medium grained Andesitic lapilli tuffs of low to medium rock strength which may be silicified to varying degrees.

2.2 Veins

The Pongkor orebodies are quartz vein systems within Andesitic tuffs, lapilli tuffs, and breccias. There are three sub-parallel main vein systems at the mine approximately 1 kilometer apart.

- Ciguha Utama vein, is orientated N 135° E/75°, approximately 0.5 kilometers in length, and extends to 150 m below surface. Orebody thickness averages 3.0 metres with average grade of 10 g/t Au and 128 g/t Ag.
- Kubang Cicau vein, is orientated N 353° E/80°, approximately 0.8 kilometers in length, and extends to 230 m below surface. Orebody thickness averages 4.0 metres with average grade of 14 g/t Au and 172 g/t Ag.
- Ciurug vein, is orientated N 344° E/59°, approx. 1.2 kilometers in length, and extends to 300 m below surface. Orebody thickness averages 10 metres with average grade of 12 g/t Au and 134 g/t Ag.
- Gudang Handak vein, is orientated N 358° E/83°, approx. 0.8 kilometers in length, and extends to 150 m below surface. Orebody thickness averages 3 metres with average grade of 7 g/t Au and 110g/t Ag.

3 GEOTECHNICAL

Before conduct the tunnel or raise, geotechnical drilling should be done to get data for geology and rock condition. It is necessary to know the rock condition along the tunnel or raise that has been planned, so the miners can do preparation of what kind of support/reinforcement method that should be applied during the activities of tunneling or raising. In this paper, we will discuss only about geotechnical activities at Ciurug Mine.

3.1 Geometry of the orebody

The Ciurug Mine is divided into 3 block of reserve such as south, central and north block, with dip 40° - 70° or average dip 54°, and average width 9.1 m (see Table 1).

3.2 Weathering

Weathering at Ciurug vein quite strong, vary between 5-25 m from the surface (in average 12 m).

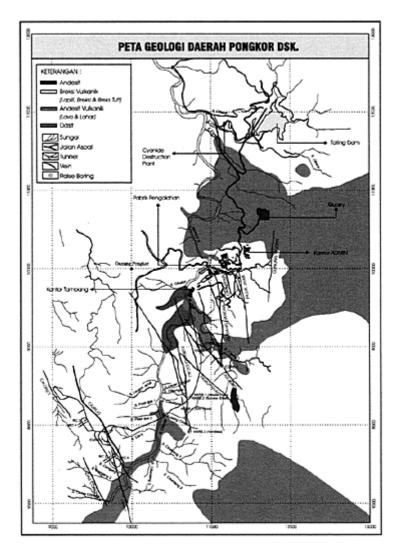


Figure 3. Geology map and ore deposits.

3.3 Rock strength

Rock strength test was done with the following result:

- Ore, with UCS test is obtaining rock strength 90 MPa (average) or between 20–200 MPa.
- Hanging wall and Foot wall, vary between 10-200 MPa.

3.4 Rock mass rating

Generally, waste rock (hanging wall and foot wall) and orebody have a RMR value with classification from poor to good rock.

4 RAISE BORING EXPERIENCE

Development activities at Pongkor divided by 2 main activities which are tunneling and raising. Raising is the activity that we can discuss in this paper. Raises, is used for ventilation, services, orepasses and for man transportation.

In year 1998, it was decided to open the Ciurug mine, which has $\pm 60\%$ total reserves of Pongkor. Contracts with Byrne Cut-RUC Mining Contractor (Australia) and Skanska (Sweden) were conducted, based on our experience at Ciguha and Kubang Cicau mine which have 7 raises using Raiseboring method (2.4 m diameter).

From geotechnical data which is taken from geotechnical logging drill in Ciurug, gave the result that the rock condition in raise location to be made (2.4 m diameter) is competent. First raise (CURB-1) was made. Pilot Hole 6" was done from surface 730 m to 515 m above sea level) to the underground, and also the reaming from bottom until finish to surface. Unfortunately, less than 1 week after, suddenly there was cavity (30 m from the surface, 25 m deep) that disconnect surface with the mine. Later on, it took more than 6 months to fix it.

The next raise (CURB-2) was made during CURB-1 being repaired. But the same cavity also happened

	Ore bloc dip(°)	ck average		Ore block average horizontal thicknes (m)		
Mining section	Area basis	Tonnase basis	Range of block dip (°)	Area	Tonnage	Range
South	53	53	38–70	7.9	9.4	3.0–14.3
Central pillar	52	52	50–55	9.7	10.2	4.6–14.2
Central	54	55	35-60	8.1	10.7	<2–18.5
North	58	55	40–73	3.6	5.2	2.0–10.4

Table 1. Dip and width of Ciurug vein.

after the reaming was done. The cavity occurred from the surface until 45 m deep. Because of its cavity condition, CURB-2 cannot be repaired.

Based on occurrence at CURB-1 and CURB-2, the management conducted a detail geotechnical investigation for the next raises (CURB-4 and CURB-5) that will be made. CURB-3 was cancelled and the management decided to move the location to another area (called CURB-2A), because the previous location is nearby to the CURB-2.

Working together with Geomin (one of Antam's SBUs), some reinforcement at CURB-4 and CURB-5 were conducted. Pilling method, that is to make 6" hole in 50 meter from surface and concreted. As much as 12 holes were done. CURB-4 then started

to do with more confidence feeling. 260 m pilot holes finished and reamer at the bottom installed. From the pilot hole we knew that at the 561–580 m and 600–608 m elevation, there was disparity of pressure at the raisebore machine. It is obvious that the possibility of weakening condition of rock have been detected. However, the reaming is still remain have to be done.

The worst case was happened. While the reamer reach elevation 630 m, there was cavity from elevation of 561–606 m. The reamer stopped because the hole was blocked to the bottom (mine), and also the reamer trapped at that block raise.

This case made the management frustrated to use the raiseboring method for making raises at Ciurug mine. From 5 raises planned to built, the same situation occurs during 3 raises being made. Even though CURB-1 have been repaired, but it also took more than 1.5 billion rupiah (approx. US \$200 thousand). The management decided to cancel the rest of raises planned.

4.1 The failure of raiseboring method

Learn from failure that happened, Pongkor Gold Mine forming a small team to evaluate failure in making raise in Ciurug mine. The team get some the failure causes:

- 1. Lack of geotechnical data. Pongkor assumed that 50 m correlation of geotechnical core drilling is relevant enough to interpret the rock condition, so when pointing the location for raise some mistakes will be appeared.
- 2. Reinforcement can only done at the surface. It is simply to reinforce the raise location by pilling method (up to 250 m), but obviously it is need a lot of money.
- 3. Raiseboring method is not flexible for Ciurug condition. When pilot hole detect the weak zone at the location, it is impossible to avoid it. Because of its characteristic of raisebore machine, the only choice after pilot holing is continue (with certain risk) or stop.

4.2 The advantages of raiseboring method

At least 3 advantages that found by the team, those are:

- 1. It needs short time to make a raise by raisebore machine.
- 2. In safety aspects, it is the most safety method to make a raise for labors.
- 3. The working area is more wider and comfortable for labors compare to working area in the underground.

5 CHANGING TO THE RAISE CLIMBER METHOD

At that moment, the team was searching the information from literatures and/or expertise in order to finish the rest of Ciurug mine's raises. Raises still have to be made, due to its purpose for ventilation and orepasses, etc.

Finally the team found the information about raise climber, and the management agreed to use raise climber method for making raises at Ciurug mine then CURB-5 was

made by that method. Through the tight training and joined by the on the job training at the location of raise to be made, Pongkor has its owned raise climber crew.

Making raise at CURB-5 was succeeded, then CURB-5 change the name become RC-1 (Raise Climber-1). After succeeded with RC-1, there are RC-2, RC-3, RC-2A (100 m long, each), RC-4, and RC-5. All are being used for service and ventilation purposes.

5.1 Advantages of raise climber method

From the result of making some raises at Ciurug mine by this, we got some advantages, such as:

- 1. Reinforcement/supporting can be done in every steps of raise progress.
- 2. If something occur to the working face and nothing support method can be implemented, raise climber can be turned up at which the condition is better.
- 3. Compared to raiseboring method which is costly relative, raise climber method which is using Indonesian labors, only spend about Rp 6 million per meter (US \$700).
- 4. It has been noted that all the sections that responsible for the job, starting from Mine Planning crews, Geotechnical crews, Surveyors and Mine Development crews are working together in every steps of raise progress.

5.2 Disadvantages of raise climber method

Pongkor's team still found some disadvantages while implementing this method.

- 1. Time consuming for 100 m raise is longer (2.4 meter per day in average).
- 2. In safety aspect, labors are facing the higher risk because they are working at the top of raise.
- 3. The size of the hole can not be made small. The minimum diameter is 2.5 m.

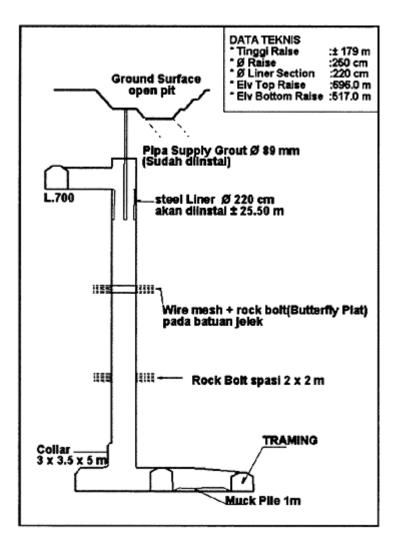


Figure 4. RC-7 design.

6 RAISE CLIMBER EXPERIENCE FOR THE RC-7 OREPASS IN CIURUG

In the beginning of year 2003, Pongkor needs an ore pass to transport ore in Ciurug from upper level to Ciurug 500 level, which has never been built before. The choice falls to make raise with raise climber method. The deep of raise is 180 meter from surface (696 m above sea level) to 517 m above sea level. It took 3 months to make RC-7. It was also decided the size of the raise is 2.5×2.5 m.

The jobs for RC-7 are following the steps given below.

6.1 Raise design

Raise design is including the length of raise, location, etc as shown in Figure 4.

After design approved by the management, co-ordinate of the location was plotted at the field (surface and the mine).

6.2 Geotechnical investigation

Geotechnical core drilling was conducted from the surface to obtain the condition of rock as long as the propose raise. It took 20 days along 228 meter by using machine LM-44. The result of geotechnical condition and also classification of rock mass (RMR) are shown on Tables 2 and 3.

6.3 Raising

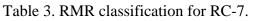
6.3.1 Preparation

Before raising get started, at the bottom of raise should be prepared, so the raise climber machine and guide rail can be installed. Some drilling & blasting occurred (if it required) in order to make the opening that the raise climber machine need (see Figure 5).

Table 2. Description of geotechnical core drill for RC-7.

No.	Depth of drilling	Description of lithology
1	0.00-1.70	Sand-tuff, green, fresh, very soft, moisture, fall to pieces and ravelled, prophylit alteration.
2	1.70–4.85	Tuffi grey become green, clay-pebble, fresh, very soft to medium hard, moistured, some of falling to pieces & break, crack loaded by iron FeOx and calcite, prophylit alteration, quartz veinlet, calcite, slicken side 60° at 42, 13 m
3	44.85– 87.40	Tuff breccias, greenes-grey, clay-cobble, inset tuff, fresh, soft to medium hard, moistured, some of falling to pieces & break, crack loaded by calcite, clay, prophylit alteration, quartz veinlet kuarsa, pirit, slicken side 30° awt 55.0 m, 20° at 56,85 m
4	87.40– 143.10	Tuff breccias, greenes-grey, clay-cobble, black silt, fresh, hard medium, dry, compact, crack loaded by calcite & some of clay, prophylit-silicification alteration, quartz veinlet and calcite
5	143.10– 228.45	Tuff breccias, greenes grey, clay-cobble, soft hard, andesit fragmented, dry, quartz veinlet and calcite, compact

No.	Lithological type	RMR	Description of lithology
1	Ι	60– 85	Tuff breccias and poly mix breccia, fresh, compact, and competent, stand up time more than 1 year, and only use scalling
2	III	30– 40	Tuff breccias, separated by several joints, soft condition and easy falling to pieces or loose
3	IV	<25	Tuff breccias, very strong alteration, condition of rock very weak,deasily falling into pieces if incured by water



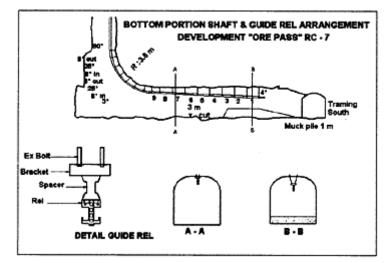


Figure 5. Bottom portion shaft and guide rail arrangement

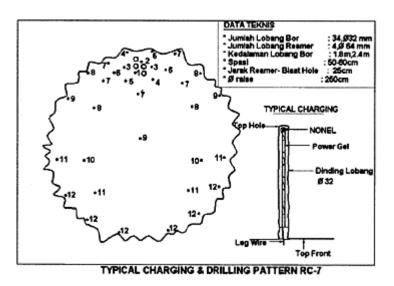


Figure 6. Drilling and charging pattern.

After the opening has been prepared, the next step is preparing electric installation, water supply and supply of compressed air approx. 6 bar pressure of minimum 6 bar (700 CFM). Also materials for reinforcement like cement, steel-liner, rock bolt, and weld mesh.

Soon after things those mention above are well prepared, continued with installation and setting of rail, the machine it self and other accessories.

6.3.2 Excavation and evaluation the progress

Even though the type of supporting has been planned for the raise, it is obvious the actual situation can be

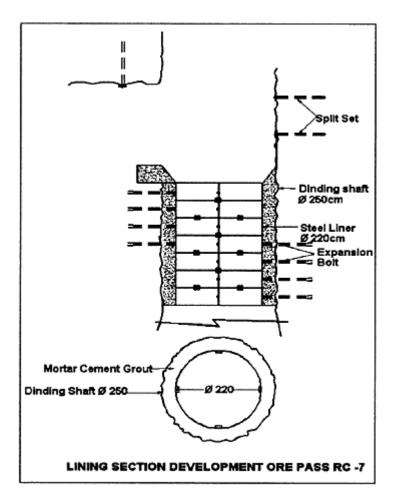


Figure 7. Raise cross section with types of supporting.

different. Using a drilling pattern and charging (see Figure 6) for every cut 1.8 m, geotechnical crew should evaluate the rock condition after each blasting. This is useful to decide the type of supporting that should install in each progress.

6.3.3 Supporting/reinforcement

Every cut of the raise that formed will be supported/reinforced by rock bolting, meshing or steel lining, and also concreting for some areas. Or, combination of those supporting method if it necessary. This kind of supporting method base on the evaluation result of geotechnical crew that mentioned above.

This cycle from drilling-blasting-supporting is continuing until the raise has reached 228 m (see Figure 7).

		7.			
No.	Elevation (m)	Depth (m)	Lithological type	Description of lithology	Supporting type
1	517.0– 604.4	0.0– 87.2	Ι	Tuff breccia, fresh,competent, stable rock	Systematic rockbolting, 2×2 m spacing
2	604.4– 608.6	87.2– 91.4	III	Tuff breccias, separate by joint which loaded by calcite,soft condition of rock, easy to loosen	Meshing with rockbolting+butterfly plate, 1.5×1.5 m spacing
3	608.6– 658.0	91.4– 140.8	Ι	Tuff breccia, greenes-grey, fresh, competent, stable, joints have potency to form scaling	Systematic rockbolting, 2×2 m spacing
4	658.0– 665.5	140.8– 148.5	III	Tuff breccia, separated by joints, with strong alteration of rock—sometimes soft alteration, easy to loosen	Meshing with rockbolting+butterfly plate, 1.5×1.5 m spacing
5	665.5– 692.0	148.5– 174.4	IV	Tuff breccia with strong alteration, very weak rock, easy to once fall to pieces if incuring water, a lot of calcite veinlet, rock very easy to collapse	In each excavation, always followed by installation of steel liner and
6	692.0– 697.0	174.4– 180.0	Ι	Tuff breccia, medium alteration, compact, separated by some joint	Systematic rockbolting, 2×2 m spacing

Table 4. Elevation and types of supporting for RC-7

The actual types of supporting that have been installed to the RC-7 can be shown at Table 4. This table consists of the elevation of raise, lithological condition and also RMR classification.

7 CONCLUSION

- 1. UBPE Pongkor has succeeded to make 6 raises by using raise climber method at Ciurug mine.
- 2. Area which is found a lot discontinuity of rock condition and weak rock, like at Ciurug mine, is good for applying raise climber method.
- 3. Each progress (cut) while making raise, the right decision for supporting can be applied soon, after it has been evaluated.
- 4. The level of success to have a raise with the rock condition as same as Ciurug mine by using raise climber method is high.
- 5. There is a good working team during the implementation of raise climber method.

6. Pongkor Gold Mine is ready to compete with other mining company/contractor in order to make a raise using raise climber method.

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