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Foreword

On behalf of the conference editorial and advisory board we welcome you to the 17th Coal Operators' Conference (Coal 2017). Since its beginning in 1998 many authors presented papers on a wide variety of topics. The availability of past papers online at http://ro.uow.edu.au/coal has provided the industry with the latest on new and current innovations, cutting edge research and developing technologies in different aspects of coal mining operations. No doubt the outcome from these annual gathering have contributed to improvement in mine production, productivity and a safe working environment.

Despite the roller-coaster coal mining industry performance over the past a year or two, the Coal Operators' Conference continues to attract a healthy number of good quality papers, but relatively fewer registrations. Nevertheless, the conference has played a significant role in bringing about progressive change. Many new ideas, innovations and safety improvements have been the centre of discussion and documentation in this conference. At present there are 638 published papers on line and documented in 16 proceedings. These papers have attracted more than 640 000 downloads since going online in 2008 and around 85 000 downloads over the past year as of January 2017. An additional 41 papers will be added online from this conference. In addition to Australia, there are also papers from Iran, Czech Republic and India.

Special thanks go to the editorial panel members, paper reviewers and the conference advisory board for their support. Thanks to Johlene Morrison for setting up the conference website, type setting the conference proceedings as well as involvement in delegate registration and administration during the conference days; Kevin Marston and Shahin Aziz, for the conference general management, financial control and logistical support. Catering was provided by the UOW Out for Lunch Pty Ltd and the Friday BBQ was hosted by the UOW's Travelling Science - Technology - Engineering - Maths (STEM) Team.

Special thanks to our sponsors, the creative hire for supplying and erecting exhibitors booth, the University of Wollongong printery staff Terry Campari for designing the conference proceedings cover page, Garry Piggott and Maria O' Hearn for printing the conference proceedings. Finally sincere thanks to all authors and participants, who are the backbone of the conference success.

All papers are peer reviewed to maintain the conference's high standing and recognition.



Professor Naj Aziz Conference executive chairman



Mr Robert J Kininmonth Conference executive co-chair

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ROOF SUPPORT AND ROADWAY SERVICEABILITY ASSESSMENT USING BEAM-COLUMN PRINCIPLES

Terry Medhurst¹ and Peter De Roma

ABSTRACT: This paper outlines the development of a roof support assessment method that takes account of differing roof conditions, effect of support type and stiffness that can be used in the strata management (TARPS) process. The proposed model is based on beam-column principles and incorporates bending, horizontal loading and shear. Estimates of roof convergence for various heights of softening (or surcharge loading) above a roadway can be obtained for a given support pressure. The model relies upon inputs from the Geophysical Strata Rating (GSR), roof bolt pull-out stiffness/load, H:V stress ratio and UCS.

INTRODUCTION

Mining at increasing depths of cover, in weaker and more variable strata conditions and with greater emphasis on optimisation of mining practice is driving the need for improvements in strata characterisation, mine planning and design. From a geotechnical perspective, the use of geophysics data was traditionally limited to UCS estimates and experienced based interpretation of sonic logs for strata characterisation. In response to this, the Geophysics Strata Rating (GSR) was developed in order to capture these basic principles traditionally applied by minesite geologists and to extend it to quantitative analysis for geotechnical applications (Hatherly *et al*, 2016).

GSR is now routinely used in several Australian coal mines for strata characterisation. GSR results can be modelled in 2D and 3D along with other parameters derived from geophysical logs such as the clay content (Medhurst *et al*, 2010). The application of these strata characterisation models for hazard planning has driven demand for design applications. With the support of ACARP funding, investigations into open cut mining (ACARP C20025), longwall caving (ACARP C20032) and roadway roof support assessment (ACARP C22008) have been undertaken. An approach was developed for longwall support assessment based on support density, stress conditions and convergence limits (Medhurst *et al*, 2014). Preliminary investigations into the application for roof support design suggested a similar principle may apply and an analytical model was developed using beam-column principles (Medhurst, 2015). Development and testing of this approach has been underway (Medhurst *et al*, 2016) and this paper discusses the latest developments and summarises the results of the most recent project (ACARP C24015).

ANALYTICAL BEAM-COLUMN MODEL

Section Properties

GSR analysis provides a continuous measure through the borehole column over the full height of the strata, as shown in Figure 1. Beam stiffness and therefore roof convergence will vary according to the distribution of hard and stiff units within the strata. GSR is based on physical measurements that are related to the composition, density and elastic properties of the strata. This means that the variation in strata stiffness within the roof beam can be estimated from GSR. A basic estimate of strata modulus has been developed where

$$E_{strata} = 1.75e^{3 \cdot GSR/100}$$
 (GPa)

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Beam analysis requires section properties such as the 2nd moment of area (I), the position of the neutral axis of the beam (y) and a measure of strata modulus (E_{strata}). Previously, the conventional approach had been chosen simply as a function of bolt length without regard to the strata. With the new approach each layer can be treated as a material of different stiffness and the theory of composite materials can be applied to obtain equivalent beam properties. The general concept is shown in Figure 2.

The position of the neutral axis over a given beam thickness can be estimated using the parallel axis theorem. Using this approach the beam is treated as being comprised of many layers of different stiffness, which is determined from the GSR analysis. The true bending behaviour of the beam is then assessed by transforming the dimensions of the beam according to the ratio of the elastic modulus of the materials. This is the standard approach for the design of composite beams in structural engineering.



Figure 1: Example of GSR analysis for a borehole



Figure 2: Definition of section properties based on transformed section

Beam-Column Formulae

For the purposes of this study a model was developed based on beam-column principles that incorporates both bending and axial loading (Timoshenko and Gere, 1963). The beam deflection due to bending is estimated using the standard method and the influence of the horizontal load (P) is treated as a multiplier (u) on both the deflection and maximum bending moment. The general formulation for a fixed end beam is shown in Figure 3.



$$\delta = (y)_{x-1/2} = \frac{5WL^4}{384EI} \frac{12(2\sec u - 2 - u^2)}{5u^4} \quad \text{where} \quad u = \frac{L}{2} \sqrt{\frac{P}{EI}} \quad \text{and} \quad I = \frac{t^3}{12}$$

Figure 3: Fixed end beam-column

The effect produced from the horizontal load (P) in the roof will increase the amount of roof deflection. Areas of high deformation/low confining stress are generally concentrated in the immediate roof. In this case there is a critical strata/beam thickness in which this deformation occurs that is not related to that defined by the bolt length, but more about the strata properties in the immediate roof. Previous studies conducted to estimate the stability of unsupported roadways found that depending on roof stiffness, there is a critical beam thickness at which failure occurs (CSIRO, 1996). This minimum thickness can be defined by a mechanism of snap-through at the mid-span as shown in Figure 4. This critical thickness (t) can then be used to estimate the multiplier (u) in the beam-column model. In other words, the additional roof convergence caused by horizontal stresses in the immediate roof is estimated based on a critical thickness for snap-thru of the beam rather than just conventional bending analysis.



Figure 4: Definition of critical thickness for beam snap-thru

By following through this analysis, it can be seen that roof deflections can be estimated from an estimate of vertical surcharge load (q), horizontal stress/load (P), GSR, and roadway width. The vertical load is simply estimated by choosing the height (or range) of softening above the roadway and the horizontal load (P) by the normal *in situ* stress regime and concentration factors about roadways.

Stress Inputs

The distribution of bending moment and therefore deflections vary depending upon the end constraints of the beam. In coal mining environments, several conditions may exist depending on stresses and/or damage in the roof beam, which can be represented by different end constraints as shown in Figure 5. A procedure has therefore been developed to calibrate beam end constraints and section properties relative to the stress and strength conditions present based on the GSR to mining induced stress ratio.



Figure 5: Beam end constraints and bending moment distribution

The influence of roadway orientation relative to the angle of the major principal stress (σ_1) also has to be considered. The standard approach is to consider the angle of the roadway relative to angle (β), which takes account of 2D orientation to σ_1 using the following

$$\sigma_{\rm R} = [0.5 * (\sigma_{\rm H} + \sigma_{\rm h}) - 0.5 * (\sigma_{\rm H} - \sigma_{\rm h}) * \cos(2\beta)]$$

Where σ_H and σ_h are the major and minor stresses in the horizontal plane.

In development, 3D stress effects need to be considered to assess stress concentrations. Following a series of 3D model calibrations, the maximum stress in the roadway can be estimated using

$$\sigma_{\rm M} = {\rm SCF} \ {\rm x} \ \sigma_{\rm R}$$

Where if virgin $\sigma_H / \sigma_V \ge 2$ then SCF = 1.2

if virgin
$$\sigma_H/\sigma_V \le 1$$
 SCF = 2 then SCF = -0.8* (σ_H/σ_V) + 2.8

A general form of the beam-column formulae can then be used to take account of the differing stress conditions in the roadway

Maximum roof deflection
$$\delta = \frac{5WL^4}{X EI} \frac{12(2 \sec u - 2 - u^2)}{5u^4}$$

Where if $1 < GSR/\sigma_M < 4$

$$X = 100e^{\left(\frac{GSR}{\sigma_M}\right)}$$

The final consideration is that for coal roof, in which correction factors need to be applied due to the difference in stresses and bending behaviour that generally exist, the following should be considered. Firstly, there is a requirement to establish conditions in which a coal roof will act as a beam. In order to maintain moment carrying capacity at the roadway corners, a minimum span to thickness ratio of 4:1 is required for a coal roof to develop an active beam. It should be noted that some judgement is required to determine that the minimum coal thickness is comprised of competent coal and roof stability is not influenced by weak planes within that horizon, such as by a series of interbedded layers of coal and thin partings or penny bands. Under these conditions, the following rules would apply:

Coal Roof

Coal thickness
$$\ge \frac{1}{4}$$
 roadway width
 $\sigma_H/\sigma_V = 1$
SCF = 1
GSR multiplied by 1.4 for coal component of roof beam GSR

In contrast to previous beam models used for support design, the aim here is to provide an analytical model that does not require estimates of cohesion, friction angle, tensile strength or other properties that are commonly difficult to measure or estimate. However in order to estimate roof convergence, the effect of the roof support must also be considered. Roof support has the effect of increasing roof stiffness, which is also usually not present in conventional beam based analysis. Hence an ability to estimate the combined stiffness of the roof strata and roof support is required.

Roof beam stiffness

In the case of a coal mine roadway, the above mentioned beam-column formulation can be used to estimate roof convergence as a function of height of softening (or surcharge loading) for a given support pressure. It is then necessary to estimate the change in stiffness of the roof beam as a function of the installed roof support. Brady and Brown (2004) provide detailed analytical solutions for rock-support interaction analysis and show that the support stiffness can be treated as two springs connected in series one being the stiffness of the roof bolt and the other the stiffness characteristics of the bolt/anchor system under pull-out load or so-called grip factor. The support stiffness is given by

Support stiffness

$$\frac{1}{k_b} = \frac{L\sqrt{S}}{N_b} \left(\frac{4L_d}{\pi d_b^2 E_b} + Q \right)$$

Where L = roadway width

S = average bolt row spacing

N_b = number of bolts or cables per row

 L_d = debonded length of bolt or cable

 d_b = diameter of bolt or cable

 E_{b} = bolt or cable modulus

Q = load deformation constant or grip factor of bolt or cable in mm/kN

The combined stiffness of the roof beam can then be treated as the strata stiffness and the support stiffness acting in parallel, which is the summation of the two. The roof beam stiffness used for the beam-column analysis is given by

$$E = E_{strata} + k_b$$

Mark *et al* (2002), provides an estimate of grip factors for fully resin grouted bolts in both Australian and U.S. coal mines. However Q values can also be obtained by short encapsulation pull-out tests or other related data. Thomas (2012) provides an outline of a series of lab-based tests on cable anchorages commonly used in Australia. Using available pull-out data estimates of load deformation factors for both bolts and cables were developed as a function of roof quality (GSR)

Load deformation factor for bolts

$$Q = \frac{1}{6.5e^{(GSR/100)}}$$

Load deformation factor for cables

$$Q = \frac{L_d}{30e^{\left(\frac{GSR}{100}\right)}}$$

Finally the pull-out capacity of the bolts needs to be estimated. The yield capacity of the bolt or cable itself is one measure, or another that includes some measure of the rock strength itself is also common, depending upon the length of anchorage. Farmer (1975) provides a simple expression based on the unconfined compressive strength (UCS) as follows:

$$P_c = 0.1 \cdot UCS \cdot \pi \cdot R \cdot L_b$$

Where P_c = pull-out capacity

R = borehole radius $L_b = \text{bond length}$

The UCS can be obtained from relevant test data or estimated from sonic velocity derived values as is often used at most operations. Depending upon the support installation, the lesser of the bolt yield capacity or pull-out capacity is used.

Bedding plane shear

Laminated and/or micaceous roof in deep or high stress zones is one of the key issues governing the selection of Trigger Action Response plans (TARPs) triggers and determining minimum installed support density in development for many Australian operations. The development of excessive shear stresses causing failure in weak strata is mostly driven by excessive roof movement due to bending in combination with horizontal stresses and/or bedding plane shear, as shown in Figure 6.

The development of steep sided shear failure surfaces often follows this initial triggering mechanism. An important indicator is therefore to establish conditions in which bedding plane shear is induced relative to the height of softening and/or imposed stresses. This requires an estimate of shear forces per metre length across the roadway. This concept is known as shear flow and is the standard approach for sizing bolts to resist shear in structures subject to bending, as shown in Figure 7.

Traditional approaches to checking for shear failure, i.e. estimating the material mass that could overcome the cohesive strength of the overlying strata generally yield high factors of safety, even in weak materials. This approach is generally too simplistic and requires a range of assumptions to

arrive at a suitable answer, unless a specific joint, fault or pre-defined surface can be used as the potential failure plane. Where such geological structure is not present, excessive yield or shearing of bolts should be checked against the maximum shear flow which occurs at the neutral axis of the beam. The beam section properties are obtained from GSR analysis. It is then possible to derive an estimate of Factor of Safety (FoS) against bedding plane shear using the shear flow calculation divided by the shear strength of the bolts.



Figure 6: Typical failure behaviour in laminated roof

The maximum shear flow (q) is given by

$$q = \frac{VQ_x}{I}$$

Where

$$V = \frac{W\rho gH}{2}$$
 shear force

 Q_x = first moment of area about neutral axis I = second moment of area H = height of softening





Figure 7: Shear flow in beams

ROOF STABILITY ASSESSMENT

Roof behaviour model

The analytical beam-column model is intended to capture the main features governing the bending and shear failure of a roadway roof beam. Distinct mechanisms such as wedge failure, or localised influence around geological structure need to be treated separately for design and within the strata management framework. The formulated roof behaviour model follows engineering statics principles in which the combined effects of varying strata conditions and the installed support are integrated into one mechanistic model. A schematic view of the roof beam model is shown in Figure 8.



Figure 8: Schematic view of roof beam model

The analytical model can be used to estimate both load and convergence behaviour of the roof beam. Roof convergence is estimated from the vertical surcharge load (W), horizontal stress/load (P), GSR, and roadway width. The vertical load is estimated by choosing the height of softening above the roadway and the horizontal load (P) by the *in situ* stress regime and the appropriate stress concentration factors about the roadways.

The output from the analytical model can be plotted as a series of graphs of roof convergence versus the installed support density. By varying the support density, the relationship between roof convergence and support load can be estimated and plotted as a curve, known as a Ground Response Curve (GRC). Several curves can then be plotted for different heights of softening and/or stress conditions. Different curves can also be plotted depending on the staging of excavation or support installation, such as after widening of a longwall installation road or installing additional support after a TARP trigger. In some cases these relationships can also be used to investigate the adequacy of the TARP's triggers themselves.

Strata-support interaction

A typical example of ground response analysis is shown in Figure 9. The intersection points between the installed support (straight line) and the GRC provide a measure of the estimated support demand and roof convergence for a given condition. This example shows that at 30 mm of roof convergence the primary bolt pattern reaches its maximum height of softening (2 m) and would attract a support load of about 35 t/m². Assuming cables were then installed at a 30 mm trigger level then the analysis suggests that the roof would stabilize at about 70 mm at a height of softening of 4 m.

A key aspect of this approach is that by introducing a convergence measure then serviceability limits can be used as design criteria. For example, a typical operational situation might be when cable support has been designed with a strength limit (Factor of Safety, FoS = 1.5), but roof convergence levels may be in excess of say 100mm leading to the requirement for further support defined in the TARPs. The issue here is the uncertainty between the relationship between roof load, the size of the failure zone in the roof and convergence, and whether further support is required.

The approach here attempts to address this issue in which a new measure is introduced based on the support pressure generated that includes the effect of cumulative roof convergence. In this case the use of serviceability criteria may provide a more representative assessment of support performance. Two methods can be used depending upon the support and site conditions. The general definition is described as the Serviceability Factor (SF) = ratio of nominal support capacity to the estimated support pressure for a given roof convergence, as shown in Figure 10. Note the difference in support stiffness for the 4, 6 and 8 bolts per metre support patterns. The ratio for the 8 bolt pattern is shown. In experience to date, a SF > 1.4 is generally recommended.



Figure 9: Example of ground response analysis



Figure 10: Serviceability Factor

An alternative approach, which may be particularly useful for primary development support is the ratio of the estimated roof convergence at the maximum bolt height to the nominated trigger level. An example based on a 20 mm trigger is shown in Figure 11. This measure may be useful for example, to assess the risk of height of softening reaching the bolt height and the need to install long tendon (cable) support.

When long tendon support is installed it is relatively rare that heights of softening reach beyond the length of cables. And if such a risk does exist, close monitoring via extensometers and TARPs control are usually implemented. In weak ground conditions, the choice of cable type, pattern density, convergence triggers and grouting requirements/triggers become critical in managing both roof stability and development productivity. From this perspective, the ability to estimate load-convergence interaction between the ground and installed support becomes a useful tool. An example is shown in Figure 12. A grouted and point anchored cable pattern assuming a height of softening of 4 m and with the cables installed after 25 m of roof convergence is shown.







Figure 12: Ground response for grouted and point anchored cable pattern

The analysis suggests that a 2 x 2 pattern of grouted cables combined with an 8 bolt pattern is about three times stiffer than the point anchored pattern. Previously no measures have been available to quantify this effect other than pull-out tests on cables. Monitored levels of roof convergence in roadways with grouted cables are often significantly greater than bolt head displacements from pull-out tests. This highlights that pull-out data alone is unlikely to be a suitable indicator of the interaction between strata and the support and its effect on roof convergence. The analytical beam-column model therefore provides an estimate of the effect of grouting on the stiffening of the roof beam. Further confirmation work is required in this area, but the model at least provides an ability to quantify the effect of grouting on roof convergence and its relationship to potential TARPs triggers.

Support analysis

Several examples of the use of ground response curves for stability assessment have been provided elsewhere (Medhurst, 2015). This approach is useful for assessing a particular set of conditions such as an area of different strata or where a roof support change is proposed (Medhurst *et al*, 2016), but can be somewhat cumbersome for design purposes. This is because the analysis is able to present many design options that require the determination of a suitable workflow, i.e. appropriate heights of softening, convergence and/or triggers need to be chosen.

In order to overcome this limitation a mathematical solution was developed to find relevant intersection points from the ground response analysis. The solution was based on a generalized logarithm for exponential-linear equations (Kalman, 2001). Design curves could then be developed between support pressure, roof convergence and height of softening for any depth, roof quality (GSR) or trigger (convergence) level.

Figure 13 shows an example of a set of design curves for a 4, 6 and 8 primary support bolt pattern (X Grade) at a depth of 300m. The plots show Factor of Safety (FoS) for bedding plane shear and Serviceability Factor (SF) versus Height of Softening (HoS) for GSR values of 30 and 50 for a roadway parallel to the major principal stress (Angle = 0). Note the change in position of the ground response curve for the different GSR values. The difference in strata conditions is also reflected in the design curves. For example at a HoS = 2 m, FoS = 1 (GSR = 30) and FoS = 1.5 (GSR = 50) for bedding plane shear for the 4 bolt pattern. Similarly, SF varies from SF \leq 1 (GSR = 30) to SF \geq 2 (GSR = 50) at a HoS = 2 m.

Using a minimum SF = 1.4, it can be seen that the maximum allowable height of softening in weak Roof (GSR =30) is about 1m for a 4 bolt pattern and about 1.5 m for an 8 bolt pattern. Whereas at GSR = 50, SF > 1.4 for all patterns at 2 m height under relatively low stress conditions (Angle = 0). The variation between different roof types is also apparent when examining the estimated roof convergence versus height of softening. This demonstrates the applicability of the 20mm trigger level often used in weak roof, since roof convergence is estimated at greater than 30 mm at 2 m height for GSR = 30. Conversely, roof convergence is estimated at between 15mm and 20 mm at 2 m for GSR = 50. Trigger levels of 15 mm are often used for primary support in stiff roof.

The effect of varying stress using the beam-column model is shown in Figure 14. In this case, the roadway angle has been changed from parallel (Angle = 0) to perpendicular (Angle = 90) as would be the case for a cut-through. Note again the change in the position of the ground response curve and the corresponding influence on the FoS and Serviceability Factor for different heights of softening. A GSR = 40 would represent average roof conditions in Australia and here again the design curves match typical support and operating practice.

Similar estimates can be undertaken for cable support. Figure 15 shows an example using a typical 2 x 60t cable support pattern installed at 1 m, 2 m and 4 m row spacing in weak roof, assuming installation after 20mm of roof convergence. The ground response curve at 4 m height of softening is shown. Applying a minimum FoS =1.5 for bedding plane shear and a minimum SF = 1.4 for bending, it can be seen that 2 m spacing would be suitable in the gateroads (Angle = 0) but inadequate in the cut-throughs (Angle = 90) at heights of softening greater than 6 m.

The analysis also suggests the influence of bedding plane shear on cable support is most distinct. For example, wide spaced cables (4 m) installed in the cut-throughs give a FoS = 1 at HoS = 4 m, suggesting failure under these conditions. The difference in the convergence plots is also apparent with an increase on average of about 25 mm from gateroads to cut-throughs. Note also the relative increase in roof convergence at heights of softening > 4 m. Experience shows that strata management issues often develop in roadways when heights of softening increase beyond 4 m.



Figure 13: Example of design curves for primary support with varying GSR



Figure 14: Example of design curves for primary support with varying stress



Figure 15: Example of design curves for cable support with varying stress

Another consideration is the timing of cable installation and its relationship to TARPs. Figure 16 shows an example in which the cables have been installed after 40 mm of roof movement and compared to the previous example of installation after 20 mm in a cut-through. Note the change in position of the ground response curve and associated change in support demand. Further inspection shows that this has no effect on FoS for bedding plane shear but a significant effect on roof bending. A significant reduction in the Serviceability Factor is shown with a commensurate increase in estimated roof convergence when the cable support is installed at 40 mm. This demonstrates the flexibility of the analysis method how it can reflect installation practice. Further examples are provided in the final research report (Medhurst, 2017) and it is intended to extend the approach to gateroad stability under longwall abutment loading conditions.



Figure 16: Example of design curves for cable support installed on different TARP

CONCLUSION

A method has been developed based on beam-column principles for use in coal mine roof stability assessment. It provides an ability to estimate both support load and roof convergence relative to the height of softening in the roof and therefore provides an ability to match analysis results against underground measurements and observations. The method relies upon inputs from the Geophysical Strata Rating (GSR), roof bolt pull-out stiffness/load, H:V stress ratio and UCS. And thus avoids the

requirement to estimate parameters such as cohesion and friction angle, which can be highly variable at roadway scale and therefore difficult to measure or estimate.

The method is aimed at providing a practical tool for support design and assessment, and has been tested at a number of Australian underground operations. It is based on conventional engineering statics and follows the basic mechanics of roof beam behaviour under bending and shear. Distinct mechanisms such as wedge failure, or localised stress influences around geological structure are not applicable to this method and need to be treated separately for design and within the strata management framework.

Estimates of Geophysical Strata Rating (GSR) are inherent to the formulation which include estimation of beam section properties, roof stiffness, influence of stress on beam fixity and installed support stiffness. An appropriate estimate of GSR is therefore critical to the reliability of the method. In some Australian mining operations, the presence of very thinly laminated strata or micaceous layers that degrade on exposure can result in overestimating the GSR when based on standard borehole geophysics measurements. Methods to correct the GSR for these conditions have been identified and are currently being tested. It is also intended to extend the approach to gateroad stability under longwall abutment loading conditions.

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EFFECTS OF FRONT ABUTMENTS AROUND MULTIPLE SEAM MINING OPERATIONS

Cihan Kayis¹ and Mehmet Siddik Kizil

ABSTRACT: Abutments form due to the redistribution of stresses around excavations. In longwall mining, these stresses can redistribute in front of the longwall panel (front abutment), over the chain pillars and intersections (side abutments) and into the goaf (goaf abutments). Front abutments are a key factor for barrier pillar design and can significantly affect secondary support performance. Front abutments for single seam mining operations can be detected using empirical methods, however these methods are not useful for multiple seam mining operations. This paper investigates the effects multiple seam mining has on the extent of the front abutment, requiring the secondary support to be installed well ahead of the retreating longwall face. This paper focuses on the examination of longwall abutments by analysing GEL extensometer data in order to identify the point where the total displacement exceeds 3 mm. Preliminary results suggest that the remnant pillars located in upper workings increase the detection distance by 125% and the goaf abutments in the upper workings decrease the total range by 50%. The secondary support framework suggests that the bolting advance rate and the longwall retreat rate should be accounted for when determining the lag distance.

INTRODUCTION

Longwall mining is a common mining method used throughout the Bowen Basin in Queensland. Multiple seam longwall mining is a relatively new concept in Australia. While extensive work has been completed in the United Kingdom and in America, these concepts may not be applicable to Australian mining conditions due to stronger and thicker coal seams (Peng, 2008). Front abutments can be used to establish the minimum width required for barrier pillars, minimising coal sterilization within pillars. The abutment can also influence the minimum lag distance required between the longwall face and the secondary support installation. It is vitally important that the secondary support is installed before the gateroad roof is subjected to front abutment effects. The abutment values can also be used to develop a database for the Bowen Basin and act as a benchmark for other future operations.

This study focuses on a multiple seam operation located within the Bowen Basin. The mine contains old bord and pillar workings located in the upper seam. The lower seam contains recently developed longwall panels. The mine has an average depth of cover of 220m with an interburden thickness of 20m. The typical upper workings which are associated with multiple seam operations include: Remnant pillars (blue), virgin ground (white), goaf areas (grey) and goaf boundaries, which are illustrated in Figure 1. It also identifies areas where the operation type is unknown, but it is expected to be goaf (green).

As the longwall retreats, the exposed roof area increases gradually. This leads to strong mine pressure manifestation on the extracting coal mass ahead of the face (Xu, Wang and Shen, 2012). An 'abutment' is the redistribution of stresses around an excavation to form high stress concentrations (Darling, 2011). The abutment pressure action is the root of various mine rock pressure manifestations (Chen and Qian, 1994).

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Figure 1: Multiple seam mine in the Bowen Basin

When mining begins at the working face, advanced abutment pressure occurs inside of the coal mass ahead of the face (Chen and Qian, 1994). The dynamic change directly reflects the roof activity, indicating roof breakage and caving (Chen and Qian, 1994). Currently, the authors are not aware of any methods or equations which link the front abutment distance and the associated multiple seam features it encounters.

Morsy, Yassien and Peng (2006) state that the probability of multiple seam interactions increases when the overburden to interburden ratio is greater than 7, the overburden depth is less than 300m and the interburden thickness is less than 30 times the mining height of the lower seam. The four controlling factors that dictate how far above and below the total extraction area the stress field changes in multiple seam mines are (Zipf, 2005): OB (Overburden)/IB (Interburden) ratio, goaf width to interburden thickness ratio, site-specific geology and horizontal stress to rock strength ratio. However, the authors are unaware of any methods used to quantify the interactions.

Current multiple seam interaction mechanisms include the pressure bulb theory and pressure arch theory. The pillars in the overlying seam using the tributary area concept (Akinkugbe, 2004) share a uniform load of overburden equally. The pressure bulb theory suggests that the load will eventually be transmitted by the pillars to the floor. The formation of a major pressure arch in longwall mining is based on two assumptions. First, the ratio of the seam depth to longwall panel width must be at a critical value so that the arch can support itself. Secondly, the gateroad pillars must be of sufficient insitu strength to support the abutment pressure (Luo, 1997). The pressure arch theory can be used to determine the minimum barrier pillar width.

Instrumentation

DETECTING ABUTMENTS

Abutments can be detected using three main methods. The first method involves performing geotechnical mapping and using pogo sticks (and other visual methods) to monitor when the stresses are thrown in front of the face. Visual observations of rib fret and roof cracking often provide a good indication of the front abutment. The second method for detecting abutments is to measure the change in stress as the abutment comes through. Numerous stress-measuring instruments are available on the market. The absolute stress can be determined using instruments which require overcoring such as Hollow Inclusion (HI) cells (Chen, 2016). However, these methods require relaxation in order to determine the stress and the data is only valid for the particular section of strata. Abutment stresses can be measured using stress changes. The most common instrument used to determine stress changes is the vibrating wire stress meter.

The final method used for detecting abutments is to measure the change in displacement concurring in either the rib or the roof of the panel. Most mines measure the roof displacement caused due to the

additional loading of the front abutment. The most common instrument used to measure roof and rib displacement is the GEL extensometer. The extensometer works by using linear potentiometers to detect strata movement (GEL Instrumentation, 2016).

Figure 2 illustrates typical data obtained from the GEL extensometers. Each colour represents a different anchor height. As shown in Figure 2, the front abutment extends approximately 55 m in front of the face in virgin ground (Shen *et. al,* 2006). Displacement data is crucial in determining secondary support lag distance. The extensometers are categorised and separated according to the upper seam workings.



Figure 2: Sample displacement data

Detecting movement

Classifying movement is essential in order to establish different movement events within the recorded data. This movement may include: creep, localised failure and front abutment movement. The data classification involves graphing the total displacement of the extensometers over the time of reading as shown in Figure 3. Figure 3 (a) illustrates that the orange extensometer shows a low total displacement occurring over a long period of time. This can be verified due when observing the density of data points.



a) Identifying creep, localised failures and front abutments



b) Identifying static movement

Figure 3: Classifying front abutment

The orange extensometer exhibits typical creep characteristics. Any creep data will need to be removed from the abutment analysis to remove the chance of data variation. Figure 3 (a) illustrates that the green extensometer increases in displacement before remaining relatively constant for a long period of time. The extensometer then reads another increase in total displacement towards the end of the readings. This extensometer illustrates the characteristics of localised failure which has resulted in the initial jump in displacement plateauing for some time before the front abutment made its way through. Any localised failure would be filtered so that the detection point was identified as the abutment rather than initial movement.

The second part of categorising the movement type is to identify the extensometers that were recording static movement and dynamic movement. This is achieved by filtering out the data values that are recording less than 3 mm of movement, based off a previous consultants' work (Golder Associates, 2014). Static movement implies that the movement may have occurred after the extensometer had been installed and was a result of the roof stresses beingredistributed. Dynamic movement is classified as any additional movement not caused by creep or stress-redistribution (i.e. front abutments). Figure 3 (b) illustrates an extensometer recording static movement. It is excluded from the final analysis as the maximum displacement is only 2.6 mm. It is important to note that increasing the static movement threshold will result in additional data being analysed for the total analysis.

FRONT ABUTMENT RESULTS

Detecting and quantifying abutments

The detection of the front abutment is relates to the point where the total displacement increases dramatically as the longwall face approaches. The front abutment detection is done by analysing the 'Displacement vs. Face Position' graph for each extensometer. Any movement below 3 mm of total displacement is rejected from the analysis. Figure 4 illustrates a Displacement vs. Face Position graph obtained from one extensometer. This particular extensometer has a front abutment detection distance of 39 m away from the longwall face. The immediate roof data point is selected as the anchor of concern because it exhibits the most displacement throughout the borehole. The total movement for each extensometer is categorised and analysed depending on the upper workings.



Figure 4: Displacement vs face position graph

Categorising abutments

The GEL extensioneters installed in the mine fall either under old remnant pillars, under goaf areas, under virgin ground or under a goaf/virgin boundary. Table 1 summarises the extensioneters installed at the mine according to their upper workings.

A total of 48 extensioneters are useable for the analysis. The ajority of the extensioneters are installed outbye of the longwall face in the belt road. These extensioneters have not recorded any data at this point in time due to the location of the production face with respect to the instruments.

Category	Number of Extensometers
Insufficient or No Data	83
Recording Static Movement	35
Recording Creep Movement	4
Under Virgin Ground	20
Under Remnant Pillars	13
Under Goaf Boundaries	6
Under Goaf Areas	9
Total	170

 Table 1: Extensometer classification from the case study

Maingate front abutments

To reduce data bias and maintain consistency, the extensometer data is categorised into maingate and tailgate results. The tailgate extensometers are exposed to additional loading due to the mining of the previous longwall panel. Basic statistics can be used to identify the confidence interval of the abutment data. A t-distribution is used for the analyses as the sample size of the data is less than 30 and the population standard deviation is unknown. Most engineering studies utilise a 95% confidence interval for the project findings (University of Columbia, 2016).

Figure 5 illustrates the box and whisker plots generated for the maingate abutments. Figure 5 clearly illustrates that the remnant pillars have the largest front abutment influence and the goaf has the least influence. The virgin ground abutments are used as a benchmark for other values. Figure 5 suggests that the remnant pillars are felt at a maximum of 90 m away from the face. This is 125% larger than similar conditions in a single seam mining operation. However, 75% of the extensometers lie within 57 m from the face compared to 21.5 m for virgin ground conditions. The presence of remnant pillars in the upper workings can result in larger barrier pillars and increased coal sterilisation. The larger influence of upper workings confirms that pressure arch theory increases the abutment loading quite substantially.



Figure 5: Maingate front abutments

Figure 5 also illustrates that the presence of goaf in the upper workings can lead to a 50% reduction in front abutment load as the maximum detection distance is at 21 m away from the face. This implies that goaf is distributing its load in all directions rather than in a uniform direction as seen for remnant pillars. Furthermore, goaf located in the upper workings will increase the side abutment loading on the pillars, which may lead to stress shadowing on the face and increased load on the chain pillars and intersections.

The smallest barrier pillar is left when the upper workings contain goaf. Figure 5 shows that the transition of goaf boundaries to remnant pillars and virgin coal can throw the front abutment more than virgin ground conditions can. It is recommended that the barrier pillar width is adjusted according to the upper workings located above the proposed longwall take off position. Table 2 encapsulates the statistical analysis performed on the maingate abutments. A 95% confidence suggests that the virgin abutments for other coal mines lie between 11 m and 22 m. This estimation lies well below those observed in the field.

Table 2 also indicates that the goaf abutments will lie somewhere between 0.6 m and 17 m from the longwall face. A front abutment influence of 0.5 m is less than a web of coal and would not occur in reality. The remnant pillar data suggests that the front abutment will be felt at a maximum of 52 m away from the face. Using these values for design could impose significant geotechnical consequences. The statistical confidence of data would increase dramatically if the sample size were larger and a z-distribution were used instead. It is recommended that other mines in the Bowen Basin use the box and whisker plot data for their preliminary studies rather than the statistical analysis.

		0			
Upper Workings	Sample Size	Sample Mean (m)	Standard Deviation (m)	Confidence Level (%)	Abutment Range (m)
Virgin Ground	16	16.72	10.02	95	11.38 – 22.06
Remnant Pillars	11	33.09	29.03	95	13.59 – 52.59
Goaf	5	9.00	6.78	95	0.58 – 17.42
Goaf Boundaries	6	27.67	13.78	95	13.21 – 42.13

Table	2:	Maingate	front	abutment	statistics
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Tailgate front abutments

Figure 6 illustrates the box and whisker plots generated for the tailgate abutments. It clearly shows that the virgin ground upper workings exhibit the greatest front abutment influence compared to the other workings. The remnant pillars only exhibit a maximum throw of 37 m compared with the 90m

seen in the maingate. However, the tailgate data suggests that the pressure arch effect is not occurring and shows that the virgin ground will have the largest throw. Using the tailgate data as a benchmark with the current data is bad practice due to the low sample size. It is possible that the last two data points are outliers if more extensometers were available for the analysis. Another reason for such large numbers occurring may be due to the fact that the extensometers have experienced prior loading due to the previous panels abutment influence. This additional loading may result in the weakening of the immediate and upper roof, resulting in a larger total displacement. It is recommended that further data is obtained from the tailgate in order to try and correlate the difference between the two and develop clear and concise conclusions.



Figure 6: Tailgate front abutments

Table 3 encapsulates the statistical analysis performed on the tailgate abutments. The analysis results in a negative detection distance due to the low sample size and the large confidence level. More data is required to improve the abutment ranges obtained using statistics.

Upper Workings	Sample	Sample Mean	Standard Deviation	Confidence	Abutment Range
	SIZe	(11)	(11)	Lever (%)	(111)
Virgin Ground	5	40.50	40.20	95	-9.41 – 90.41
Remnant Pillars	2	31.00	8.49	95	-45.24 – 107.20
Goaf	2	14.25	13.08	95	-103.30 – 131.80
Goaf Boundaries	3	27.00	19.08	95	-20.39 – 74.39

Table 3: Tailgate front abutment statistics

The observed maingate data will be used for the secondary support framework because of the higher confidence embedded in the data and the fact that roof support will be installed in the belt road (maingate) rather than in the return airways (tailgate). Overall, statistics is not a good method for result justification because of the large sample sizes required to make concise conclusions. It is recommended that the secondary support system is developed based on the maximum remnant pillar abutment throw as these conditions have been observed in the mine.

COMPARING EMPIRICAL METHODS

The extent of the front abutment can be directly related to the barrier pillar thickness (Kendorski and Bunnell, 2007). Many empirical tools exist to predict the minimum barrier pillar thickness for single seam mining operations. However, the accuracy and applicability of these methods may not apply to a multiple seam mining operation.

(1)

Peng and Chiang's method

Peng and Chiang (1984) found that the depth of the front abutment extent (D) can be a function of the square root of the coal seam depth (H) and a empircal constant which can be shown in Equation 1:

$$D = 5.13 * \sqrt{H}$$

Dunn's Rule

Dunn's rule was developed in the United Kindgom where the coal seams are much thinner and weaker. Kendorski and Blummel (2007) suggest that Dunn's rule only considered the depth of the coal seam (H). The equation has been converted from an imperial system to a metric system, as summaried in Equation 2:

$$D = \left(\frac{((3.28084*H) - 180)}{20} + 15\right) * 0.3048$$
⁽²⁾

Pennsylvania mines inspectorate rule

The Pennsylvaia Mines Inspectorate Rule was developed in America where the geology of the coal seams varies from those seen in Australia. The method has been converted from an imperial approach to the metric system. Kendorski and Blunnell (2007) show that this method incorporates the seam depth (H) and the roadway height (T) which can be summarised in Equation 3:

D = (20 + (4 * (3.28084 * T) + 0.1 * (3.28084 * H)) * 0.3048(3)

Pressure arch method

The pressure arch method is an adaption for seam interaction. The pressure arch method assumes that the distressed zone caused by excavations forms a dome shape, as illustrated in Figure 7.



Figure 7: Pressure arch method (Luo, 1997)

Kendorski and Blummel (2007) show that the pressures arch method, in looking at the seam depth (H), can be summarised as in Equation 4. This method was converted from an imperial system to a metric system.

$$D = (2.625 * \left(\frac{(3.28084*H)}{20} + 20\right)) * 0.3048$$
(4)

Case study results

The mine site seam parameters from the case study required for the empirical analysis include (Lines, 2015):

- Seam Depth of 220 m; and
- Roadway Height of 3.5 m

Figure 8 compares the empirical extent equations with the abutment distances seen in the mine. Figure 8 illustrates that Dunn's Rule exhibits the smallest barrier pillar width of 12.8 m for all of the methods. The maximum single seam abutment detected by the extensometers occurs at 40 m away from the face, suggesting that a 40 m barrier pillar is required to reduce the impacts of additional loading. Using Dunn's Rule for future designs may lead to pillar failure and additional loading in the life of mine mains. Figure 8 suggests that the optimum method, which could be used for the Bowen Basin single seam operations, is the Pennsylvania Mines Inspectorate rule.





This empirical rule resulted in a minimum barrier pillar width of 42.1 m which is slightly larger than the virgin abutments detected. The pressure arch method results in a minimum width of 35 m; however, it does not consider the effects of remnant pillars in the upper seam. The largest barrier pillar width obtained from the empirical analysis is the Peng and Chiang method, with a minimum with of 76.1 m. However, this thickness is smaller than the abutment influence distance. The Peng and Chiang method would be suitable for a multiple seam operation because the pillar will behave elastically until the yield pressure is reached due to the retreating longwall. The barrier pillar width should be adjusted according to the upper workings that lie above the longwall take off.

SECONDARY SUPPORT FRAMEWORK

The secondary support framework is used to decipher the minimum required distance (i.e. lag distance) between the production face and the bolting crew. Currently, the lag distance primarily depends on the front abutment influence and does not consider any production data. Figure 9 illustrates a schematic of the definition of the lag distance. Figure 10 illustrates the flowsheet used for the secondary support framework.



The belt road requires sporadic or continuous cable support to ensure the risk of roof falls and localised failures are minimised. The key factors involved with the framework development include: Front abutment influence, longwall retreat rate and bolting advance rate. The first step of the framework is to define the operational assumptions used at the mine.



Figure 10: Framework flowsheet

The following framework assumptions are based off the mines current operations:

- 20 cable bolts can be installed and tensioned per shift;
- there are two secondary support shifts per day;
- the bolts are installed in cycles;
- one cycle involves five shifts of bolting and tensioning (done together) (2.5 days);
- one cycle involves two shifts of grouting once the five shifts of bolting and tensioning have been completed;

- one cycle involves two shifts for the grout to reach an acceptable strength after pumping; and
- one cycle is 4.5 days long (i.e. nine shifts).

The density and spacing of the bolts is determined by the geotechnical engineer. Generally, the density of the bolts is increased when the bolting crew reach intersections, work under remnant pillars and goaf areas from the upper seam and when the geology of the area changes due to roof quality and discontinuities. The mine currently deploys four different bolting patterns, ranging from one cable bolt every 2 m to three staggered cable bolts every 0.75 m.

The density patterns are represented by purple, red, yellow and blue colour patterns. Figure 11 illustrates the bolting codes used at the mine.



Figure 11: Cable bolt patterns

Figure 12 illustrates the daily advance rates achieved using the different bolting patterns. Other mines will contain similar plans with different bolting densities installed in cut throughs and other troublesome areas determined by the geotechnical engineer. The bolting patterns and bolting plans can be used to derive a representative bolting rate for the entire main. Table 4 summarises the secondary support distances measured using AutoCAD. Some of the roadways require sporadic bolting and are considered along with the intersections. Table 4 also summarises the total unmberof days required to bolt the respected regions, based on the operational assumptions. This is done by dividing the distances by the bolting advance rate.



Figure 12: Cable bolt plan
Code	Distance (m)	Support Installation (Days)
Yellow	956	25
Red	471	25
Blue	1 723	67
Purple	233	45
Total	3 385	120

Table 4: Case study cable bolt statistics

Using a total of 3,385 m and 120 days, a representative advance rate (bolting and tensioning) for the Case Study of 28.3 m/day is achieved. It is important to note that this value is site specific. This rate corresponds to a support density slightly higher than what is required for code blue. The next step of the framework requires that grouting is incorporated into the bolting time. This can be determined by dividing the product of the bolting and tensioning cycle time (A) with the bolting and tensioning advance rate (B) over the total cycle time (C), which can be summarised in Equation 5.

Bolting Rate (Total) =
$$\frac{A^*B}{C} = \frac{2.5^*28.3}{4.5} = 15.72 \text{m/d}$$
 (5)

The results show that a representative advance rate (bolting, tensioning, grouting and setting) of 15.72 m/day can be achieved for the case study. The next step of the framework requires determining the longwall retreat rate (m/day). The longwall retreat rate is obtained by the panel deputy measuring the maingate and tailgate chainage each shift and logging the distances on their statutory reports. These rates can be used to obtain an average longwall retreat rate which is site specific.

The next step of the framework is to identify the front abutment influence caused by the longwall operation. Figure 5 illustrated that the abutments detected in the maingate range from 21 m to 90 m. From an engineering perspective, it is recommended that the maximum abutment is used as it will provide the largest factor of safety. Therefore, a front abutment of 90 m will be representative of the mine. However, other mining operations should use the abutment distances detected for their site. The single seam abutment distances established in this paper may be used if no data is available.

Four different scenarios have been devised in order to cover the most extreme cases and the most realistic cases for the mine. These are:

- Scenario 1: Slowest bolting rate and fastest longwall retreat rate (Bolting Worst Case);
- Scenario 2: Case based off the current bolting rate and longwall rate (Realistic Case);
- Scenario 3: Fastest bolting rate and slowest longwall retreat rate (Bolting Best Case); and
- Scenario 4: Fastest bolting rate and fastest longwall retreat rate (Optimum Case).

Table 5 summarises the bolting advance rate (m/day) and longwall retreat rate (m/day) for each scenario. The data used for these scenarios is obtained using Equation 5 and Figure 12.

Table 5 shows that for Scenario 1, the longwall retreat rate is greater than the bolting advance rate. This implies that the longwall will catch up to the bolting crew if substantial distance is not left. This will require the longwall panel length as one of the inputs. If the longwall retreat rate is less than the bolting advance rate, then the minimum distance required is the equivalent distance the longwall will travel for a bolting cycle. Equations 6 (Method 1) and 7 (Method 2) represents the two scenarios as IF scenarios as IF statements:

Scenario	1	2	3	4
Longwall Retreat Rate	>10.83	<15.72	<28.89	<28.89
Bolting Advance Rate	10.83	15.72	28.89	28.89

Table 5: Framework scenarios

IF LR > BR,
$$x(m) = P - \left(\left(\frac{P-L}{LR}\right) * BR\right)$$
 (6)

IF
$$LR \le BR$$
, $x(m) = L + (LR * BC)$ (7)

In Equation 6 and 7, 'x' represents the minimum lag distance required between the longwall face and the bolting crew (m), 'P' represents the panel length (m), 'L' represents the abutment distance (m), 'LR' represents the longwall retreat rate (m/day), 'BR' represents the bolting advance rate (m/day) and 'BC' represents the total bolting cycle time (days).

Equation 6 shows that different parameters used in order to determine the required distance. The bolting cycle is the time taken between the initial bolt installation and the time for the grout to fully harden in the last bolt. Some mining operations may utilise a different secondary support approach. The IF statement should be adjusted according to the condition, then the minimum required distance will be dependent on the specific panels. It is assumed that the average panel length for the mining operation is 2200 m. Table 6 encapsulates the minimum distance required for all four scenarios, utilising Equations 6 and 7.

Table 6 shows that the minimum required distance for current mining practices is 119.7 m. The lag distance ranges from 105 m to 719 m, with the minimum distance increasing significantly for scenario 1. This phenomenon occurs due to the fact that the longwall will catch up to the bolting crew if significant distance is not left. Figure 13 illustrates the required distance based on varying the abutment distance.

Scenario	1	2	3	4
Longwall Retreat (m/day)	>10.83	<15.72	<28.89	<28.89
Bolting Advance (m/day)	10.83	15.72	28.89	28.89
Abutment Length (m)	90	90	90	90
Panel Length (m)	2200	-	-	-
Distance Required (m)	718.90	119.70	104.95	159.43

Table 6: Required distances



Figure 13: Required distance vs abutment distance

Figure 13 illustrates that all four scenarios show linear trends. The trend line equation generated represents the current trends being observed at the mine. A minimum distance of 29.3 m is required due to bolting and production data. Disregarding the dynamic inputs of the framework may lead to the cable bolts not being used to their maximum capacities due to early loading of the roof from the front abutment. Figure 14 illustrates the required distance for the mine based on the longwall retreat rate.



Figure 14: Required distance vs longwall retreat rate

Figure 14 shows that the required minimum distance can be selected based on how the longwall is performing. The required distance increases tremendously once the longwall rate surpasses the bolting rate. Figures 13 and 14 clearly show that the required distance is higher than those being currently used throughout the mine. Using an abutment distance of 90 m will contain an incorporated factor of safety. It is recommended that the mine uses a lag distance of 120 m to account for the new parameters. It is important to note that this model does not account for intersections and cut through bolting. In the event that the longwall is faster than the bolting rate, it is recommended that an additional crew is used to perform the secondary support installation in the cut throughs in order to not to interfere with the primary crew operations.

CONCLUSION

In conclusion, the effects of the front abutment will be amplified by up to 125% if the longwall is retreating under remnant pillars. The effects will be dampened by up to 50% if the longwall retreats under upper seam goaf areas. The empirical equations suggest that the Pennsylvania Mines Inspectorate method is the most accurate at predicting single seam abutments. However, the empirical methods failed to predict longwall abutments under old bord and pillar workings. Peng and Chiang's method tends to overestimate single seam abutments; however, it underestimates the maximum abutment throw experienced in the mine.

The secondary support requirements suggest that the lag distance is a function of the bolting advance rate, longwall retreat rate, panel length and the front abutment. The lag distance is significantly amplified if the longwall retreat rate exceeds the bolting rate. The limitations of the support distance are that it does not incorporate the time required for intersection bolting and the curing time for the grout is developed around industry rules of thumb. The framework can be adjusted to incorporate parameters which are more applicable to certain sites. It is recommended that the abutment values obtained from the extensioneters are compared with other sites from the Bowen Basin and around Australia to ensure that the data is valid and looks reasonable.

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IMPACT OF BEDDING PLANE AND LAMINATIONS ON SOFTENING ZONE AROUND THE ROADWAYS – 3D NUMERICAL ASSESSMENT

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ABSTRACT: When the distributed rock stress around the roadways exceeds the strength of the rock, the rock is failed and a softening zone is formed. Roof deformation developed in the roof and ribs of the roadways are highly controlled by the depth of softening zones. The rock failure process starts from a point ahead of the face and grows into the roof, floor and ribs by advancing roadway. The maximum stress that can be transferred through the failed rocks would be equal to its residual confined strength. Therefore, rock stress is moved above failed zone and will create new failure zone if it is higher than the confined strength of rock at that depth. This process continues until the confined strength of the rock becomes higher than stress components. Bedding and lamination planes play a big role into the failure pathway of rocks around roadways. The thickness of softening zone is significantly influenced by the shear and tensile strength of bedding planes and laminations. This paper presents a 3D numerical assessment of the bedding and lamination planes impacts to the forming and extension of the softening zones. It highlights the requirements for better characterisation of bedding and lamination planes for reliable simulation of roadways.

INTRODUCTION

Bedding planes and laminations are two common terminologies for sedimentary rocks. The stratification, which is the main sedimentary structure, is defined as layering of sediments throughout the sediment deposition. The stratification can be divided into two groups of bedding and lamination on the basis of the strata thickness. The lamination term is used when the strata thickness is less than 1 cm and it represents a sequence of fine layers. For the strata thicker than 1 cm, there are several sub-groups for description of bedding structure (Table 1). The bedding plane can be easily identified when the lithology of the adjacent beds is different (Figure 1). When the lithology of the adjacent beds is same (Figure 2), the bedding plane is hard to be recognised (Campbell, 1967).

	Terminology	Strat Thickness [cm]	
	Very thick-bedded	> 100	
	Thick-bedded	30 – 100	
Bedding (Bed, Beds)	Medium-bedded	10 – 30	
	Thin-bedded	3 – 10	
	Very thin-bedded	1 – 3	
Lomination (Lomina, Lominata)	Laminated	0.3 – 1	
Lamination (Lamina, Laminate)	Thinly laminated	< 0.3	

Table 1: Bedding and lamination	terminologies for sedimentary	v rocks (Campbell, 1967)
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Figure 1: Bedding plane when lithology of the adjacent beds is different



Figure 2: Bedding plane when lithology of the adjacent beds is same

Shear strength of bedding planes and laminations can be determined by triaxial test or direct shear test. For direct shear test, the cores recovered from vertical or angled boreholes can be used. This test is more reliable for the shear strength of the partings. The following reasons limit the direct shear application for the tight bedding planes and laminations:

- Test setup: it is difficult to align the shearing direction correctly while casting the core into the upper and lower boxes.
- Loading boundary condition: providing pure shear loading condition is arguable because of developing tension stress on one side of the sample.

Considering the above limitations, the triaxial test is the best option to determine the shear strength of tight bedding planes. The bedding plane and laminations are acting as a weak plane within the sample. The Mohr-Coulomb failure criteria for a sample including a weak plane with an angle of β with the direction of maximum principal stress (σ_1) is as follow (Jaeger and Cook, 1979):

$$\sigma_1 - \sigma_3 = \frac{2(c + \sigma_3 tan\varphi)}{(1 - tan\varphi cot\beta)sin2\beta}$$

where, σ_3 is minimum principal stress (confining stress), *c* is cohesion of intact rock, and φ represents the friction angle of the intact rock. For $\varphi < \beta < \pi/2$, the failure occurs along the weak plane. The graph of the maximum principal stress components against confining stresses represents the failure criteria for the weak plane.

The core samples for the tri-axial test on bedding plane are drilled at an angle to the bedding planes (Typically 30°). This drilling angle provides samples with angle between the bedding plane and maximum principal stress of 60° which guaranties the failure along the bedding plane. Figure 3 shows typical failure curves for the bedding planes obtained from direct shear and triaxial tests.



Figure 3: Typical strength characteristics of bedding planes and weak planes (SCT Operations 1995).

IMPACT OF BEDDING PLANES AND LAMINATIONS ON SOFTENING ZONE

In reality, roadway excavation removes the *in situ* confinement of the surrounding rocks. In considering the triaxial behaviour of rock, less confinement results in lower strength and more possibility for the redistributed stress to exceed the strength of rocks. When the new stress components exceed the strength of rocks, rocks fails in different modes. Apart from the failure of the intact rock, stress can exceeds the strength (tensile or shear strength) of the discontinuities such as bedding planes, lamination planes, and rock joints. Therefore combination and extension of all these failure modes create the softening zones around the roadway.

The historical field measurement showed that there is a relationship between the magnitude of roof deformation and the height of softening (Figure 4). The height of softening is defined by the height to which significant failure into the roof is occurring.

In this paper, 3D numerical simulation is implemented to study the impact of the bedding plane and the laminations to the extension of the softening zone around the roadway. FLAC3D (Itasca, 2013) was used for 3D simulation and SCT's rock failure simulation fish code (Gale, 1998) was extended to 3D and was applied for this analysis. In this numerical modelling, 1 m of rock is excavated as each excavation sequence and model is run to detect induced failure zone. This process is repeated to reach the desired excavation length of roadway. The length of modelled roadway should be enough to eliminate the boundary condition impacts. In failure simulation process, the strength of the material is determined on the basis of the confining pressure, friction angle and cohesion of the material, bedding and joints. The general strength characteristics of the rock materials in the intact and post failure range are presented in Figure 5. The model simulates rock fracture and stores the orientation of the fractures. Shear fracture, tension fracture of the rock, bedding plane shear and tension fracture of bedding is determined in the simulation. Failure modes obtained from this simulation are presented by different numbers (Table 2).



Figure 4: General relationship between roof displacement and height of softening (SCT Training Manual).



Figure 5: Generalised strength characteristics for rock units (SCT Training Manual).

Geological and geomechanical characterisation of Bulli seam was used for the numerical simulations presented in this paper. Variation of the compressive strength (UCS) of roof and floor strata is presented in Figure 6. These figures show that the UCS of roof and floor strata varies between 10 - 80 MPa and 22 – 60 MPa respectively. Coal seam thickness is 3 m and simulated roadway has 5 m width and 3 m height. Rock support elements include eight roof bolts with 2.1 m length and two rib bolts with 1.2 m length. Depth of simulated roadway is 500.

Code	Failure Mode	Code	Failure Mode	
0	No Failure	7	Tension fracture reactivation in tension	
1, 2	Shear fracture reactivation in shear	8	Tension failure of bedding and reactivation in tension	
3	Bedding shear and reactivation			
4	Tension Failure or reactivation in shear	9	Shear failure in intact rock	
5,6	Tension reactivation of shear fractures			

Table 2: Numbers which used to present various failure modes



UCS (MPa)

Figure 6: UCS profile through the 3D modelling boundary.

A medium-bedded stratum (based on Table 1) was assumed for the first 3 m of the roof and the floor. The stratum becomes thick-bedded for the rest of the model. In addition to this, two stress orientation scenarios of; 1) a maximum horizontal stress perpendicular to the roadway axis and, 2) a 45 degree between maximum horizontal stress and the roadway axis were modelled.

The results of 3D modelling in the form of failure modes and the extension for two stress orientation scenarios are shown in Figures 7 and 8. It can be seen that the bedding shear failure and reactivation in shear are the failure modes, which control the extension of the failure zone for both stress orientation scenarios. The height of failed zone is approximately 5.3 m and 4.2 m for perpendicular stress and 45 degree stress respectively.



Figure 7: Failure modes and extension for model having bedding planes – stress perpendicular.



Figure 8: Failure modes and extension for model having bedding planes – stress 45 degree.

From Figure 8, the shear failure through the rock in the roof and floor strata is skewed toward left rib due to the stress orientation impact. The 3D modelling correctly simulated the guttering failure mechanism induced by stress concentration at the left corner of the roof and the floor.

To assess the impact of the bedding planes and laminations on the extension of the softening zone, above modelling were repeated without having bedding planes impact. In this regard, the tensile and shear strength of the bedding planes were increased to eliminate the potential for tensile and shear failure along the bedding planes. This condition can represent a massive unit in roof and floor of the roadway. The failure modes and the extension of the failure zones for two stress orientation scenarios are presented in Figure 9 and 10.

By eliminating the potential for bedding planes failure, the stress redistribution causes shear and tensile failure through the rocks. The extension of the failure zone for both stress orientation scenarios were reduced to 2 m. This height of softening zone is approximately 38% and 48% of the height of softening zones for the models including the bedding planes.

The comparison of the failure modes in Figure 8 with Figure 10 reveals another major impact of the bedding planes to the behaviour of the roadways. The following failure pathways to explain the failure mechanism of roadways within the stratified and laminated rocks correctly obtained from this 3D numerical modelling include (Figure 11):

- 1. Stress concentration at the left corners of the roadway face breaking bedding planes close to the roof and the floor. This stress concentration also causes shear failure through the rock units.
- 2. By advancing of the face, the confinement provided by the face rocks decreases. Then the combination of rock shear failure and separation of the bedding planes pushes the stress concentration into the overlaying units.
- 3. Now, the stress concentration within the overlaying units causes shear failure through the rock and bedding planes. This pushes the stress further toward the roof.
- 4. This process is repeated until the confined strength of the rock and bedding planes exceeds the concentrated stress and the failure process is stopped.

Above failure pathway can explains the mechanism for the visible guttering formed in roadways. This mechanism shows that the following conditions are required to form the guttering:

- 1. The elevated stress exceeding the confined strength of the rocks, bedding planes and laminations.
- 2. The angle between roadway axis and maximum horizontal stress is as low as 10° and as high as 80°.
- 3. There are bedding planes or laminated rocks in the roof to interact with the shear failure of the rocks.

When a massive unit exists in the roof or floor, the shear failure of rock cannot be skewed toward the roadway ribs. The introduced 3D numerical modelling has the capability to simulate all this failure pathways with higher reliability.



Figure 9: Failure modes and extension for model having no bedding planes – stress perpendicular.



Figure 10: Failure modes and extension for model having no bedding planes – stress 45 degree.





CONCLUSION

The impact of the bedding planes and laminations on the failure mechanism and extension of the softening zone around the roadways were studied by 3D numerical modelling. The results of this study show that bedding planes control both the extension of the failure zone and the overall behaviour of the roadways. Bedding planes or laminated rocks in roof increases the height of softening zone in the roof. It also increases the extension of the failure in the floor and raises the potential of floor heave. For cases with maximum horizontal stress having an angle 10-80 degree with the roadway axis, bedding planes skewed the shear failure of the rocks toward the corresponding rib. These conditions facilitate guttering formation in the roof. This study shows that the introduced 3D numerical modelling has the capability to simulate the impact of the bedding planes and laminated rock with high reliability. It provides a powerful tool for stability analysis and roadway reinforcement design.

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CONNECTIVITY OF MINING INDUCED FRACTURES BELOW LONGWALL PANELS: A MODELLING APPROACH

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ABSTRACT: Gas make into active longwall panels is an important issue in ventilation and gas drainage design. A method of simulating the mining induced fracture network and associated increase in hydraulic conductivity is a necessity for improved mine design, hazard management planning and gas drainage efficiency. This paper identifies and illustrates the key components in determining the connectivity of lower gas sources to an active goaf. Computer modelling identifies the formation of cyclic fractures that form below the longwall face and extend down back below the goaf. These cyclic fractures form when the stress conditions are high enough and the strata properties allow for shear failure to extend down through the strata. The mining induced fracture formation and stress redistribution creates increased hydraulic conductivity of the floor strata below the active goaf. The stress redistribution and fracture volume also reduce the pore pressure below the goaf, allowing gas desorption to occur from lower seams. The combination of gas desorption and increased hydraulic conductivity allows gas connectivity from gas sources below the seam to the active goaf. A monitoring program at a NSW mine as part of ACARP Project C23009 allowed for preliminary validation of the concepts illustrated from the computer modelling. Preliminary field gas flow measurements are within the range of connectivity expectations based on rock failure modelling of longwall extraction. This report presents the first validation results for the modelling approach presented in this paper. Further results from ACARP Project C23009 on optimisation of gas drainage will follow in future publications.

INTRODUCTION

Gas make into coal mines is an increasingly important issue with the increasing depth of coal mines. Together with the longwall method of extraction and with increasingly wider panels, an understanding of gas make into the actively mined seam from adjacent seams is necessary. Coal mine safety and regulations require adequate gas management plans incorporated into the mine design and operations. Coal mine gas is generally controlled through mine ventilation and gas drainage. To optimise ventilation and gas drainage design, an understanding of the volume and rate of gas make into the mine is the key element.

There are robust methods for measuring gas content, volume and desorption characteristics in coal, however there are less robust methods of estimating the volume of gas that makes its way into the mine ventilation system from adjacent seams. This paper investigates the connectivity of mining induced fractures that form below longwall panels and their potential to facilitate gas make into the active goaf from coal seams below.

Computer modelling software, FLAC 2D, was used to simulate fracture formation below longwall panels. These models illustrate the concept and nature of mining induced fracture formation that facilitates gas flow from lower seams to the active goaf. The computer models are also used to assess connectivity between the lower seams and the active goaf by providing vertical hydraulic conductivity estimates for gas drainage and ventilation assessments. FLAC 3D was also used to simulate gas make and gas drainage in simplified three dimensional models.

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Previous studies have shown good correlation of the SCT modelling approach using FLAC2D for rock failure and overburden caving about longwall panels, in particular in Gale *et al.* (2004) and Gale and Tarrant (1997). This paper is the initial validation for computer modelling of gas connectivity to lower seams using SCT's approach.

Preliminary results from a recent field monitoring program are included in this report to provide an initial validation of the modelling concept for gas connectivity to lower coal seams. The field monitoring program was part of ACARP project C23009 (Gale and Rippon, 2016) and was conducted at NSW mine.

MINING INDUCED FRACTURING BELOW A LONGWALL PANEL

Often in gassy mines the recorded gas make is more than the volume of gas that the operational seam can produce. This alone indicates that there must be an alternative gas source from surrounding seams. Although increased mine gas volumes can occur from interaction with overlying strata, the underlying seam interconnection is assessed in this study.

Fracturing below the longwall panel can be distinct and highly connective or may not form vertical connective fractures at all. Parametric studies conducted using computer simulation of longwall extraction show that the mining induced fractures below a longwall panel vary in nature depending on depth/vertical stress, horizontal stress, lithology, geotechnical properties and the presence of low cohesion units.

The models show the connective mining induced fractures forming at the face and extending down back underneath the goaf. These fractures are cyclic in nature and occur about every 5 m. The length and conductivity of these fractures depends on the factors mentioned in the points above.

Some scenarios are such that the stresses aren't high enough to produce distinct cyclic fractures, or failure is focussed on shear failure of weak planes, and the fractures don't extend through the weak unit. In other scenarios the presence of weak units or bedding planes increases the deformation and associated connectivity within the floor. Alternatively in other scenarios the fractures form, however the strain, dilation or stress transfer through the fractures is such that the connectivity through the fractures is near *in situ*.

Figure 1 shows a model of a Queensland coal seam example at 300 m depth showing the cyclic nature of the mining induced fractures. Figure 1a shows the mode of failure as a mixture of bedding plane shear failure, shear failure of intact rock, and reactivation of cleat or joints. The mixture of the different modes of failure creates the shear fracture that extends down below the longwall goaf.



a) Mode of failure.

b) Incremental shear strain.



Figure 1b shows the incremental shear strain, which highlights movement on the fracture in addition to the initial rock failure. The movement and dilation of these shear fractures would potentially increase their aperture and connectivity. Likewise, fractures with no to little shear strain are not likely to have large apertures and associated higher conductivity.

The two dimensional longitudinal cross section model presents the fractures as they would form under the stress conditions in the centre of the longwall panel. A cross section model across the panel width shows mining induced fractures extending from the panel edge down below the goaf. Figure 2 illustrates the modelled panel edge fractures below the extracted longwall panel.



Figure 2: Computer model of panel width cross section showing panel edge floor fractures.

The three dimensional concept of the mining induced fracture system that forms below a longwall block is presented in Figure 3. In the centre of the panel there are cyclic fractures forming approximately every 5 m. A long fracture forms along the length of the panel by interconnection of panel edge floor fractures and cyclic fractures.



Figure 3: Model interpretation of mining induced floor fractures.

The concept consists of a panel edge fracture that forms on the panel edge and extends below the goaf at approximately 5 degrees from vertical. These panel edge fractures are observed in the panel width cross sectional models as observed in Figure 2. The cyclic fractures that form in the dynamic longitudinal models, observed in Figure 1, are repeated along the length of the panel and join up with the panel edge fractures.

A plan view concept of how both of these two dimensional model observations would manifest in three dimensions is presented in Figure 4. This concept shows how the cyclic face fractures are

envisaged to curve around and form the panel edge fractures. The curvature of the face fractures to form the panel side fractures is similar to what is observed on the surface where subsidence cracks form in a cyclic nature in the dynamic high strain zone at the face. These subsidence cracks then curve around to meet the panel edge high strain zone cracks.



Figure 4: Plan view of cyclic floor fractures below panel.

FACTORS INFLUENCING FRACTURE CONNECTIVITY AND GAS MAKE

Although the mining induced floor fractures may have formed below an active panel, there are a number of factors that contribute to the floor fractures facilitating gas flow from the lower seams to the active goaf. These factors consist of fracture geometry and dilation, stress, pore pressure/gas desorption, gas volume and goaf loading.

The physical aperture and connectivity of the fractures plays a key role in the overall hydraulic conductivity of the fracture system. Fractures that form along the high strain zone from the face to down under the goaf may not have enough strain to dilate and increase the fracture aperture. This is dependent on the stress and rock properties described earlier. Generally higher stress environments would be expected to produce increased deformation and increase the effective fracture aperture.

Additionally, individual fractures that form along these shear strain zones may not connect together to form a continuous fracture plane. The mode of failure model in Figure 1 shows that the shear fracture zone is made up of many bedding fractures, joints or cleat or shear of intact rock. For low strains, these individual fractures may not connect to form a continuous fracture plane as depicted in the concept model.

The physical attributes of the cyclic floor fractures can however be assessed through analysis of computer models for the site specific scenarios to determine the mining induced fracture network. This mining induced fracture system is then hydraulically stressed to determine the hydraulic conductivity of the seam floor fractures.

Stress plays a significant role in both the conductivity of the mining induced fractures and the pore pressure of underlying seams.

Longwall extraction significantly changes the stress environment with stress redistribution occurring about the excavation, caved overburden and fractured floor strata. Figure 5 shows the modelled vertical stress redistribution about a longwall face. Key features of this stress redistribution are:

- Increased abutment stress at the face also contributing to cyclic shear fractures in the floor strata;
- Reduction in stress behind the face this area creates low stress transfer across fractures, causing increased conductivity, and also reducing pore pressure in underlying seams causing gas desorption;

 Increased vertical stress at some distance behind the face – goaf reloading increases vertical stress in the centre of the panel increasing pore pressure and reducing horizontal conductivity.



Figure 5: Modelled vertical stress distribution about a longwall face.

A reduction in vertical load below the longwall panel is often enough to reduce the pore pressure of close underlying seams to below desorption pressure, thus making gas available for migration. Figure 6 shows an example of depressurisation below a longwall panel, where the fractures are observed to directly reduce the pore pressure for 50-80 m below the seam. This includes the reduction in pore pressure of an underlying seam at approximately 60 m below the operational seam.



Figure 6: Modelled pore pressure about a longwall face.

The pore pressure contours in Figure 6 also show an increase in pore pressure due to goaf loading at approximately 80-100 m behind the face for this example. This increase in loading could increase the pore pressure to above gas desorption pressure to limit gas supply. If the pore pressure is increased to a pressure below desorption pressure, the gas flow would be expected to be reduced below the goaf due to the reduction in pressure gradient.

The volume of gas in the coal directly influences the volume of gas that has the potential to make its way to the active goaf.

Goaf loading reduces hydraulic conductivity in the horizontal plane, with little impact on the vertical conductivity. A significant increase in horizontal stress would be required to reduce the vertical

conductivity. The reduction in conductivity of the horizontal planes reduces the area from which vertical fractures can source gas volumes.

COMPUTER MODELLING ASSESSMENT OF CONCEPTS

The computer models have proposed a concept that suggests the mechanism for gas flow from lower seams to the active goaf via a network of existing fractures and mining induced fractures. The models have also identified stress redistribution about the longwall panels and its impact on pore pressure and fracture conductivity.

Further manipulation of the computer models provides assessment of the connectivity of the fracture system below the active panel through assessment of vertical hydraulic conductivity. The models also provide a means to assess different sensitivities that are expected to influence the conductivity and gas flow. Specific elements assessed to further understand the connectivity concept include:

- 1. Hydraulic conductivity of the fracture system in the floor
- 2. Impact of goaf loading on horizontal and vertical conductivity
- 3. Gas flow from fractured strata without drainage boreholes
- 4. Gas flow from fractured strata with drainage boreholes

Assessment of these elements is presented for a case study at a NSW mine. The field program of work was designed as part of ACARP project C23009 where assessment of the gas connectivity concept through measurement of pore pressure and gas flow in cross measure gas drainage wells was conducted. The working seam has cross measure gas drainage holes intersecting four lower seams. The actual gas flows and calculated average hydraulic conductivity are discussed in the following section. The site specific characteristics from the case study are used in this section to address the different key elements of the connectivity concept.

The hydraulic conductivity of the floor fracture system is determined by modelling flow through the fracture network as a result of a hydraulically stressed model. The vertical conductivity is a result of flow through horizontal and vertical fractures that form the fracture network. The result of this is a detailed vertical hydraulic conductivity section through the model fracture network. Figure 7a shows the vertical conductivity of the case study model where the conductivity of the cyclic fractures is in the order of 5×10^{-6} m/s. For practical purposes, the conductivity is best presented as an average conductivity in order to estimate average gas flows for a given area, where the average vertical conductivity for the 50 m behind the face is $1 \times 10^{-7 \text{ to -8}}$ m/s.

An important factor in understanding connectivity is to understand the impact of goaf reloading on the conductivity and resulting gas flow. The horizontal and vertical conductivity for the case study have been teased apart to address this element of connectivity. Figure 7 shows a comparison of modelled vertical and horizontal conductivity for the area of stress relaxation behind the face and the area experiencing goaf reloading.



a) Vertical conductivity.

b) Horizontal conductivity.

Figure 7: Modelled conductivity results for the case study mine.

The horizontal conductivity shows a significant reduction from 1×10^{-2} m/s to 1×10^{-6} m/s due to the additional vertical goaf load, however the vertical conductivity remains relatively unchanged at approximately 1×10^{-7} to $^{-8}$ m/s due to low variation in horizontal stress. This implies that although the vertical conductivity is similar, the gas source may be reduced due to the reduction in lateral connection and surface area to the gas source. In the same way that goaf reloading occurs at a distance behind the face, the goaf loading in the panel width section is only in the panel centre, leaving a stress relaxation zone at the panel edges. Figure 8 shows the vertical stress distribution across two adjacent panels showing the goaf loading effects in a zone in the panel centre with relaxed zones either side.



Figure 8: Vertical stress distribution across panel widths.

A simpler equivalent mass model based on the fracture characteristics of the rock failure models was created in FLAC 3D to assess gas flow from the thickest underlying seam into the active goaf. Scenarios were modelled without goaf drainage wells and with wells at 20 m and 100 m spacing. Model variables also included wells intersecting one or two horizontal fractures, and different down hole well pressures to determine the range and characteristics of expected gas flows. The coal desorption pressure was modelled at 1.5 MPa and the drainage holes had a diameter of 45 mm.

The results show that for the scenario without drainage holes, flow is predominantly on the horizontal partings in the coal, before intersecting with the sub vertical mining induced fractures. The results for the flow model without gas drainage wells are presented in Figure 9a. The trend in Figure 9a represents the total gas flow volume for a 50 m x 50 m area of gas make into the goaf.



without drainage holes.

Figure 9: Modelled gas make for the case study mine.

For the models with gas drainage wells, flow is concentrated along horizontal partings and sub vertical mining induced fractures, until the drainage well is intersected. For boreholes that intersect the lower coal seams, direct flow from the seam is also observed from the borehole.

The flow outputs from the drainage wells for different model scenarios are presented in Figure 9b. The modelled scenarios include the flow for expected end members for a single hole over a 50 m x 50 m area and, per hole, for three holes draining the same volume. Both scenarios are run for a maximum and minimum downhole pressure of 600 kPa and zero back pressure.

The peak drainage well flows were modelled to be in the order of 500 L/s, where the flows then quickly reduced in an exponential manner and were mostly reduced by approximately 20-60 days from the onset of drainage. The rapid decrease in flow rate is due to:

- 1. A reduction in pressure gradient caused by a reduction in gas volume and pore pressure due to gas being drained from the seam
- 2. A reduction in pressure gradient due to gas drainage from a farther distance from the drainage well.

These preliminary models do not include goaf loading. The models indicate the range and trend of anticipated gas flow into a gas drainage well. The models indicate that for the case study longwall extraction, peak flows of up to 50 L/s would be anticipated, with flows quickly reducing to low flows of less than 100 L/s in 2-25 days.

MODEL VALIDATION WITH EARLY FIELD MEASUREMENT RESULTS

A large field monitoring program for the case study has captured a significant amount of information to investigate gas drainage and borehole efficiency. The field program of work is far more involved than the results discussed in this paper, however this paper shows the preliminary results that illustrate initial validation of the fracture system and connectivity concepts presented in this paper. The case study field trial monitored cross measure gas drainage wells and found that the peak flows varied depending on the measures that the boreholes intersected. Figure 10 shows the flows measured in the boreholes that drilled through the different horizons at various stages of retreat and in relation to the number of days the wells were on line.



Figure 10: Gas drainage well flow monitoring from case study mine

Boreholes that intersected the upper two seams had peak flows up to approximately 100 L/s, while drainage holes that were drilled through the four seams showed peak flows up to 250 L/s. The increase in flow volumes is generally consistent with the increase of volume of gas available for desorption.

The peak flows were measured to be less than the anticipated maximum 500 L/s. It is however noted that if the initial flow in a drainage well is not observed, and the first flow measurement is taken even a couple of days later, then the peak flows would not have been recorded. The actual initial peak flows on onset of drainage are expected to be greater than the measured peak flows.

The modelled trends for single hole and 25 m hole spacing are plotted on Figure 10 for models where the hole intersects only the two upper seams and where the hole intersects all four seams. The data is generally in the range of the modelled results and follows similar trends of reduction over time. The modelled hydraulic conductivity estimates have therefore produced flows within the order of magnitude of measured flows.

The gas flow from the drainage wells shows a trend of reducing flow exponentially over time. The time at which the flow significantly reduces in all holes varies between 20 and 60 days. This is consistent with the modelled flow reduction trends.

The flows from the gas drainage holes presented in Figure 10 show a rapid reduction in flow within 100 m of longwall retreat and significantly reduced flows after 200-300 m of extraction. The rapid reduction in flow at 100 m of retreat suggests that goaf loading may play a role in reducing the area of high horizontal conductivity, thus reducing the gas source for the borehole. This is consistent with the modelling results showing a reduction in horizontal conductivity from goaf loading.

Both the magnitude of peak flows and timing of flow reduction measured in the case study are within the range of modelled expectations. This indicates that the model concept of fracture formation, estimated hydraulic conductivity and flow reduction over time is consistent with the measured results.

The modelling of gas flow for different well spacing provides an interesting insight into interpretation of effective drainage wells. On initial assessment, the drainage holes with lower flows may appear to be compromised holes with less effective drainage, however the model results indicate the opposite.

The modelling results show that a number of adjacent effective drainage wells can produce lower flows than drainage wells acting as a single well due to compromised adjacent wells. A high flowing

well may indicate adjacent compromised wells. This leads to further work in understanding the optimal spacing of drainage wells.

FORWARD WORK PROGRAM

This paper presents the preliminary results as part of a much larger program of work for ACARP Project C23009. Further results from ACARP Project C23009 on optimisation of gas drainage will follow in future publications. The ACARP study intended to form a basic understanding of gas make into active goafs and gas drainage wells in order to optimise gas drainage. A number of key areas for further work include:

- 1. Investigate the impact of turbulent flow on gas flow rates
- 2. Investigate the impact of gas desorption rates on gas drainage flow rates
- 3. Assess gas drainage borehole efficiency in regards to optimal spacing

CONCLUSION

Gas make into active longwall panels is an important issue in ventilation and gas drainage design. A method of simulating the mining induced fracture network and associated increase in hydraulic conductivity is a necessity for improved mine design, hazard management planning and gas drainage efficiency. Computer modelling illustrates the formation of mining induced fractures below a longwall block and their connectivity for gas make into an active goaf. A number of elements have been identified as key factors in the connectivity of lower seam gas to the active seam.

- 1. Cyclic mining induced fracture formation below longwall panels
- 2. Stress redistribution
- 3. Pore pressure reduction
- 4. Vertical hydraulic conductivity

Preliminary field gas flow measurements measured at the case study mine are within the range of connectivity expectations based on rock failure modelling of longwall extraction. This report presents the first validation results for vertical conductivity estimates for the modelling approach presented in this paper.

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INSIGHTS INTO THE MECHANICS OF MULTI-SEAM SUBSIDENCE FROM ASHTON UNDERGROUND MINE

Ken Mills¹ and Stephen Wilson²

ABSTRACT: Examples of subsidence monitoring of multi-seam mining in Australian conditions are relatively limited compared to the extensive database of monitoring from single seam mining. The subsidence monitoring data now available from the mining of longwall panels in two seams at the Ashton Underground Mine (Ashton) provides an opportunity to significantly advance the understanding of subsidence behaviour in response to multi-seam mining in a regular offset geometry. This paper presents an analysis and interpretation of the multi-seam subsidence monitoring data from the first five panels in the second seam at the Ashton Underground Mine. The methods used to estimate subsidence effects for the planned third seam of mining are also presented.

Observations of the characteristics of multi-seam subsidence indicate that although more complex than single seam mining, the subsidence movements are regular and reasonably predictable. Movements are constrained within the general footprint of the active panel. They are however sensitive to the relative panel geometries in each seam and to the direction of mining. In an offset geometry, tilt and strain levels are observed to remain at single seam levels despite the greater vertical displacement. At stacked goaf edges tilt and strain levels are up to four times greater. Latent subsidence recovered from the overlying seam has been identified as a key contributor to the subsidence outcomes. Some conventional single seam concepts such as angle of draw and subcritical/supercritical behaviour are less meaningful in a multi-seam environment.

INTRODUCTION

An environmental assessment of subsidence impacts and consequences to the natural and built surface and sub-surface features is required for all mining approvals in New South Wales (NSW). The prediction of subsidence effects to inform environment and infrastructure risk assessments is one of the first steps in the approval pathway. Prediction of subsidence effects for single seam coal mining in NSW has a basis in analysis of the extensive empirical subsidence monitoring databases by Holla and others (Holla 1987, Holla 1991, Holla and Barclay 2000) for the Newcastle, Western and Southern Coalfields. The Holla approach provides for the estimation of the main subsidence parameters; vertical displacement, tilt and strains.

No such databases currently exist for multi-seam mining. Case studies of longwall mining in multiseam environments by Li *et al* (2007 and 2010) and MSEC (2007) have generally involved irregular longwall geometries or a combination of longwall and pillar extraction (bord and pillar) areas. While the effects of multi-seam mining to tilt and strain levels have been discussed, the recommendations of these previous studies by Li et al (2007) are mainly restricted to methods of estimating the magnitude of vertical displacement.

At Ashton, a conservative approach based on 85% of the combined seam or mining heights (after Li *et al* 2010) was used to estimate vertical subsidence. In the absence of published guidelines for multiseam tilts and strains, estimations of these subsidence parameters were made using the Holla (1991) approach for single seam mining in the Western Coalfield with inputs of vertical subsidence derived from the Li *et al* subsidence approach.

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Site specific monitoring data now available from the mining of the first five longwall panels in the second seam at Ashton, indicates that the Li *et al* (2010) method provides a generally conservative estimate of vertical displacement at this site. The Holla (1991) approach overestimates tilt and strain for an offset geometry but underestimates tilt and strain for a stacked geometry. The monitoring undertaken at Ashton allows these estimates to be refined for particular geometries.

The Ashton site is unique when compared to other multi-seam sites for a number of reasons. These include:

- modern, reliable mine plan records
- no areas of irregular pillar extraction (bord and pillar mining)
- no potential for small pillars (or 'stooks') to fail and contribute to risk of pillar run or pillar creep
- only longwall mining in a regular, parallel layout with substantial remaining chain pillars
- gradually increasing overburden thickness toward the west, so that the depth to panels increases with each subsequent panel
- longwall panels with different starting and finishing positions and goaf edge geometries that enable a range of mining scenarios to be studied.

BACKGROUND AND SITE DESCRIPTION

Ashton Coal Operations Pty Ltd (ACOL), owned by Yancoal Australia Ltd, operates the Ashton Underground Mine near Camberwell in the Hunter Valley of NSW. The mine operates via modified development consent for the Ashton Coal Project (ACP). The mining approval covers the mining of four seams. In descending order these seams are the Pikes Gully (PG), Upper Liddell (ULD), Upper Lower Liddell (ULLD) and Lower Barrett (LB).

Figure 1 is a site plan showing the outline of the longwall panels for the PG, ULD and ULLD seams with the position of subsidence monitoring lines on a topographic map of the surface area.

The first longwall in the uppermost PG Seam commenced extraction in 2007. A series of eight longwall panels have been mined in the PG Seam. The longwalls in the second seam (ULD Seam) started in 2012. ACOL commenced mining the fifth longwall (Longwall 105) in the ULD Seam below the previous PG Seam panels in 2016.

The panels in each of the four seams were originally approved to be arranged in a regular, parallel, stacked (superimposed) geometry. The layout design has been modified to an offset (staggered) geometry to reduce subsidence impacts and take advantage of reduced stress conditions during roadway development.

In this offset geometry the 1^{st} (PG) and 3^{rd} (ULLD) seams are superimposed. The 2^{nd} (ULD) and 4^{th} (LB) seams are also superimposed but with a 60 m offset, to the west, relative to the other two seams.



Figure 1: Site Plan showing longwall panels within the PG, ULD and ULLD seams with subsidence monitoring lines superimposed onto a 1:25,000 topographic series map of the area.

All longwalls panels form a void that is nominally 216 m wide. All inter-panel chain pillars are 24 m wide (coal rib to rib). The panels are aligned in a general north-south direction. The longwall face retreats from south to north. The panel sequence is from the east to the west. The naming convention for corresponding panels is Longwall 1 in the PG Seam, Longwall101 in the ULD Seam and Longwall 201 in the ULLD Seam.

The overburden depth to the PG Seam increases from around 40 m in the northeast corner of Longwall 1 to around 180 m in the southwest corner of Longwall 7. The interburden thicknesses are generally around 35-40 m for the PG to ULD seams, 20-25m for the ULD to ULLD seams and 35-45 m for the ULLD to LB seams.

The mining height for each seam is approximately $2.5 \text{ m} \pm 0.3 \text{ m}$. Mining heights are either a function of the seam thickness or the practical operating range of the mining equipment.

The strata dip moderately to the west at around 1 in 10. This strata gradient is generally greater than the gradients of the surface topography.

The surface topography above the mining area is dominated by a steeply rising ridge line adjacent to Glennies Creek in the east from which the ground slopes west toward Bowmans Creek and the Hunter River to the south.

The longwall mining area is bounded by consideration of the subsidence impacts and consequences to both the natural and built surface and sub-surface features. The main features include the New England Highway and infrastructure in the north, Glennies Creek to the east, the Hunter River to the south and to the west, a combination of Bowmans Creek, Bowmans Creek Diversions and adjacent mining operations.

OVERVIEW OF SUBSIDENCE MONITORING

Subsidence monitoring began with the commencement of PG Seam longwall operations in early 2007 and has continued above all the panels mined since then. Subsidence monitoring above the ULD Seam Longwalls 101-104 commenced in 2012.

The monitoring data provides significant insight into the mechanics that drive the magnitude and the distribution of subsidence movements in the multi-seam environment at the site. Effects such as:

- difference in behaviour between strata that is undisturbed by previous mining and strata that has already been subsided
- recovery of latent subsidence from the overlying seam
- particular behaviour that occurs above stacked goaf edges
- the effect of mining direction on subsidence above stacked goaf edges.

Subsidence monitoring and observation

A comprehensive subsidence monitoring program involving high confidence three dimensional (3D) survey measurements has been in place since the inception of longwall mining at the Ashton site.

For the PG Seam longwalls, some 35 monitoring lines have been installed and regularly surveyed. These subsidence monitoring lines are aligned both along the panels (longitudinal) and across the panels (transverse). Additional 3D monitoring has also been conducted at other surface features or infrastructure.

For the ULD Seam mining a series of around 10 longitudinal lines have been established adjacent to the PG Seam lines at both the southern and northern ends of panels. Measurements have regularly been recorded on these ULD Seam monitoring lines to supplement further surveys on the cross panel lines previously installed for the PG Seam mining.

The main cross panel line (XL5) extends over all the southern longwalls in both seams. This line was resurveyed along its full length at the completion of the PG Seam longwalls and again after the fourth longwall in the ULD Seam.

Subsidence behaviour observed for the PG Seam mining has been consistent with the subsidence behaviour expected in panels of supercritical width and within the range as indicated by the Holla (1991) Western Coalfield guidelines.

The results of the ULD Seam monitoring show that subsidence behaviour falls into two categories depending on the relative geometries of the mining in the two seams. In most areas, subsidence behaviour can be categorised as general background subsidence behaviour with tilts and strains of similar magnitude to those observed in the PG Seam. Where the goaf edges in the two seams are located directly above each other, a different style of behaviour is apparent.

General subsidence behaviour

Figure 2 shows recent subsidence monitoring results from XL5 Line, the main cross-panel subsidence line over all the southern panels.

Measurements at the ACP site indicate single seam mining of undisturbed ground causes surface vertical subsidence of generally about 50-60% of the seam thickness mined.

Where panels in two seams overlap in the offset geometry, mining a second seam below already disturbed ground causes maximum cumulative subsidence from mining both seams of about 62-72% of the combined mining thickness. The general incremental subsidence for the second seam mined is in the order of 72-83% of the mining height.

Near goaf edges in the overlying seam, maximum incremental subsidence is observed to increase as a result of what is referred to as latent subsidence; subsidence which did not occur during mining of the first seam owing to the support provided by nearby chain pillars but is recovered when the second seam is mined. Subsidence as high as 92% of the second seam mining height is apparent when latent subsidence occurs, but the magnitude of this additional subsidence is not a function of the seam mining height in the lower seam so representing it as such is somewhat misleading.

Remote from pillar and goaf edges the maximum values of tilt and strains are typically of a similar or lower magnitude to the tilt and strains measured for the first seam mined despite the greater total vertical subsidence. The maximum values of tilt and strain are typically less than the maximum calculated assuming single seam mining conditions but occasionally increase to the same magnitude as those measured during mining in the PG Seam. This behaviour is thought to be due to a general softening effect of the multi-seam mining and the difference in behaviour between strata that is undisturbed by previous mining and strata that has already been subsided (disturbed or modified).

Behaviour at stacked goaf edges

A different behaviour is observed in areas where overlying goaf edges interact to form a stacked goaf edge. At these stacked goaf edges and particularly when the deeper seam has undercut the upper seam by a distance about equal to the interburden depth between seams, transient tilts and strains have been recorded as being much higher than elsewhere.

Figure 3 shows subsidence monitoring results from the northern end of Longwall 102 where this panel mined directly under an existing goaf edge in the PG Seam so that the goaf edges in the PG and ULD Seams were momentarily stacked directly above each other before being undercut.

At a stacked goaf edge where the lower seam is mined into solid from below an existing goaf in the upper seam, a double goaf edge is formed. Maximum tilts in these areas are observed to be about double the maximum general background levels. Horizontal strains are observed to peak at about four

times the background levels observed more generally along the panel. These maxima are observed when the goaf edge in the upper seam is undercut by a distance equal to about 0.7 times the interburden thickness.

Where mining in the lower seam has continued beyond the stacked goaf edge and beyond the undercut these high values associated with the stacked goaf edge decrease towards the more general levels measured elsewhere along the panel.

The presence of the transition from goaf to solid at a goaf edge created by mining in the overlying seam appears to focus additional subsidence movements associated with mining the deeper seam into the same location. The strains and tilts reach a maximum when the lower seam has mined past the upper seam goaf edge by a distance of about 0.7 the separation between the two seams, or where the caving of the goaf at the lower seam longwall face intersects the goaf edge in the upper seam.

Figure 4 illustrates the retreat of the ULD Seam longwalls under the PG Seam goaf edge/ solid coal and how the subsiding strata interact with the overlying goaf edge as the panel retreats. In effect, the presence of the pre-existing goaf fractures from the PG Seam mining acts as a preferred separation point so that further deformations from the ULD Seam mining are concentrated at these fractures temporarily elevating the tilt and strain levels.

Where the lower seam is mined as a single seam situation and merges under an overlying goaf, a variation of stacked goaf edge is formed. The nature of the subsidence profile in this circumstance is significantly different with a large block above the start of the overlying panel subsiding en masse as the existing goaf edge is mined under in the lower seam. The subsidence parameters of tilt and strain are of similar magnitude to single seam mining.







Figure 4: Sketch illustrating the mechanism that concentrates the strata movements during mining in the ULD Seam at the same location as they were concentrated during mining in the PG Seam.

Figure 5 illustrates the geometries involved and shows how the disturbance caused to the ground by mining each of the two longwall panels in two different seams leaves a triangular wedge of largely undisturbed ground above the start of the PG Seam longwall. This triangle of rock subsides gradually en-masse as mining in the underlying ULD Seam progresses.



Figure 5: Sketch illustrating the mechanism by which a wedge of undisturbed strata subsides as the ULD Seam longwalls mine under the start of the PG Seam longwalls.

The direction of mining in the second seam under an existing goaf has a significant influence on the surface effects that develop. Mining from a goaf under solid leads to a stacked goaf edge that produces very high tilts and strains and much higher than the general background values. Mining from solid to under a goaf produces an en masse subsidence effect with tilts and strains that are comparable to general background levels.

Incremental vertical subsidence above the ULD Seam chain pillars between Longwall 101 and Longwall 104 is approximately 200-300 mm at a depth to the lower seam of around 140 m. This subsidence is much higher than the elastic strata compression of 20-30 mm observed above the chain pillars formed by mining in the PG Seam in the same area. This is considered to be a result of compression of the disturbed ground above the lower seam chain pillar and the reduced stiffness of this ground from the previous episode of mining.

Horizontal movement

The magnitude, direction, and form of horizontal movements are consistent with the cross-panel horizontal movement observed during mining of the PG Seam longwalls. The influences of the offset

geometry and latent subsidence recovered from the PG Seam are seen in the profile as a regular pattern of incremental horizontal movements associated with the ULD Seam mining.

Horizontal subsidence movements measured above the first four longwalls in the ULD Seam are typically in the range of 20-30% of the vertical subsidence. There is a strong similarity in the characteristics and distribution of horizontal subsidence movements between these longwalls indicating a consistent mechanism driving the horizontal movements. There is also a strong influence of strata dilation in the development of horizontal movements causing a general shift in an uphill direction.

Figure 6 illustrates the mechanics of the upslope horizontal movements over longwall panels at Ashton.



Figure 6: Sketch illustrating the mechanics of upslope horizontal movement at Ashton.

The incremental long panel horizontal movements are characterised by movement toward the approaching longwall face, followed by movement in the reserve direction after the longwall face has passed.

The maximum total horizontal movement above each of the first four ULD Seam longwalls is typically about 0.8 m. This total horizontal movement is dominated by the cumulative cross panel movements which generally reaches a magnitude of about 0.7 m to the east (i.e. uphill) at a location near the western edge of the overlap between each of the ULD Seam longwalls and the corresponding PG Seam longwall above.

Multi-seam subsidence zones across the panel width

Observations of the vertical subsidence profile show a number of distinct zones of ground behaviour through the interaction of the mining geometries of each seam. These zones include:

- subsidence outside the current panel but over previous goaf
- subsidence remote from chain and abutment pillars in both seams
- latent subsidence adjacent to overlying chain pillar
- subsidence above undermined chain pillar.

The individual components of the increment of subsidence are shown in Figure 7. Each of these components has different characteristics:

Subsidence outside the current panel but over previous goaf

Beyond the vertical subsidence profile for the first seam mining, the second seam mining causes only a small amount of additional subsidence due the undermining of the softer disturbed (modified) ground.

Subsidence remote from chain and abutment pillars in both seams

For the first seam of mining the vertical displacement is around 1.4 m or 55-60% of seam mining height. After the second seam is mined an increment of 1.8m or 70-75% of the second seam mining height is realised. The total cumulative vertical subsidence is 3.3 m or 65-70% of the combined seam mining heights.

Latent subsidence adjacent to the overlying chain pillar

Increases in the incremental vertical subsidence profile for the second seam of mining are seen adjacent to both sides of the overlying chain pillar. The second seam mining causes a maximum incremental vertical subsidence of 2.2 m. This subsidence represents 85-90% of the second seam mining height. However, this subsidence includes latent subsidence recovered from the upper seam that was previously restricted by the supporting effects of the nearby chain pillar. Latent subsidence is not a function of the mined seam height in the lower seam.

Subsidence above undermined chain pillar

For the first seam mining the vertical displacement is less than 0.1 m as expected for a supercritical geometry at shallow overburden depth. After the second seam is mined the increment of vertical subsidence is 1.4m or the same as the subsidence in the first seam in areas remote from the chain pillar or panel edges. A cycle of mining one longwall panel causes undisturbed ground to subsidence by a given amount of 50-60% of seam thickness whether it is the first seam mined under undisturbed ground or the second seam mined under undisturbed ground above a chain pillar in the first seam.



Figure 7: Individual components of incremental subsidence profile.

MULTI-SEAM INCREMENTAL SUBSIDENCE OBSERVATIONS

The subsidence monitoring above Longwalls 101-104 (and the preliminary results from Longwall 105) indicates that for an offset mining geometry, the maximum subsidence can now be estimated with

reasonable confidence in the multi-seam environment at the ACP site. The subsidence profile is also relatively predictable once the specific mechanics of the interaction of the two seams is recognised.

Figure 8 shows the incremental vertical displacements and cross panel horizontal movements for the five ULD Seam longwalls (Longwalls101-105) mined to date. The profiles are overlaid relative to each ULD Seam panel edge. The subsidence behaviour observed indicates:

- a regular, repeatable form, with a general smoothing and reduction in peak values with increasing overburden depth
- the maximum vertical and horizontal movements occur substantially within the footprint of the active panel
- movements over the previous panel are insignificant for all practical purposes
- the influence of the recovered latent subsidence from the PG Seam extends over the softer, disturbed ground of the next panel due to the location of pillars in this offset panel geometry.



Figure 8: Incremental Vertical Subsidence and Cross Panel Horizontal Movements and for LW101, LW102, LW103, LW104 and LW105.

IMPLICATIONS FOR SUBSIDENCE PREDICTIONS

It is clear from this data that multi-seam subsidence presents a number of challenges for describing the subsidence behaviour. In a single seam mining environment, the subsidence behaviour is consistent with and largely controlled by the mining geometry in the seam that is mined. In a multi-seam mining environment, the presence of previous mining in an overlying seam means that the subsidence behaviour is no longer simply a geometrical function of the seam being mined, but rather a sometimes complex interaction of the geometries in both seams. Furthermore impact assessments need to recognise the effects of earlier mining.

At some locations, the incremental subsidence may be a higher proportion of the seam thickness in the second seam mined when subsidence associated with mining in the first seam is recovered. This recovering of latent subsidence is particularly evident around the edges of the first seam panels where the overburden strata was supported on the chain pillars and abutments following mining in the first seam. When the second seam mines under the chain pillars and other abutment edges, the strata above the first seam chain pillar and abutment edges is disturbed and the supporting effect around the edges is lost. The ground above the edge of the first seam chain pillar and other abutments subsidence by an increased amount that includes subsidence that did not occur during mining in the first seam. This latent subsidence increment has both vertical and horizontal components.

An understanding of both the magnitude and distribution of this latent incremental subsidence is useful for estimating the likely cumulative subsidence profile for any future mining. The maximum subsidence is not simply the addition of all the maximum increments, but rather the addition of the individual incremental profiles for each seam including the areas of latent subsidence.

Figure 9 shows the latent vertical subsidence sections (adjacent to the overlying chain pillar edges) in the ULD Seam incremental vertical subsidence profile across all longwall panels.



Figure 9: XL5 monitoring line subsidence - Latent vertical subsidence sections (adjacent to overlying chain pillar) in ULD Seam incremental subsidence profile.

Figures 10 and 11 show examples of the latent vertical subsidence recovered at goaf edges from monitoring along the panels, including a temporary stacked goaf edges near the northern end of the panels.

The concept of an angle of draw determined purely as a function of overburden depth becomes somewhat less meaningful in a multi-seam mining environment because of the influence of previous mining and the interaction of overlying geometries. Beyond the solid goaf edge in the outermost seam, angles of draw appear to have a similar magnitude to those in a single seam mining environment. Where there is an existing goaf, the concept of an angle of draw becomes less meaningful not only because of the previous subsidence that has occurred but also because of the influence of latent subsidence.



Figure 10: Subsidence movements observed on Line LW102CL1 during mining of Longwall 102.
IMPROVED PREDICTION METHODOLOGY

The method used to predict subsidence for the ULD Seam longwalls was originally based on 85% of the combined seam mining thickness (after Li *et al* 2010). The guidelines presented by Holla (1991) for the Western Coalfields were used to estimate tilts and strains. These approaches appear to still be valid based on the comparison of past predictions with subsequent measurements, but there is clearly room for refinement now that more multi-seam subsidence monitoring data is available.

The measurements from Longwalls 101-105 indicate that the maximum cumulative vertical subsidence was less than 75% of the combined seams extraction heights. The maximum incremental vertical displacement due to the recovery of latent subsidence adjacent to abutment or pillar edges was higher than 85% of the ULD Seam mining thickness but this latent subsidence is subsidence that did not occur during mining in the first seam. Latent subsidence effects need to be determined separately from the general body subsidence.



Figure 11: Subsidence movements observed on Line LW102CL2 during mining of Longwall 102.

The monitoring data from Longwalls 101-105 indicates that the use of 85% of the combined seam mining heights is a reasonably conservative approach to estimating maximum cumulative subsidence. A refinement to this estimating method is used to predict the total subsidence profile for the proposed ULLD Seam mining. This method estimates the incremental subsidence profile i.e. the subsidence associated with mining in the ULLD Seam, additional to the actual cumulative subsidence that previously occurred due to mining in the PG and ULD Seams, rather than a percentage of the combined seams mining heights, as the cumulative subsidence from the PG and ULD seams is known.

A conservative approach has been adopted for estimating the incremental subsidence for the proposed ULLD Seam longwalls as this will be the first occasion in Australia where longwall mining extracts three coal seams in a multi-seam operation.

The estimated incremental subsidence profile for Longwalls 201-204 is based on 85% of the planned mining height of this seam plus an allowance for the amount of latent subsidence to be recovered from around pillar and abutment edges in the overlying seam (or seams). The allowance for latent subsidence is somewhat interpretative but is consistent with the site parameters (similar seam mining heights) and improved understandings of multi-seam subsidence gained from the monitoring conducted to date. A small additional allowance is then applied for any remaining uncertainties around the extent of the multi-seam interactions as overburden depth increases and for any other variations in mining heights and depth of cover along the length of the panels.

Maximum strains and tilts are sometimes of interest on an incremental basis, and sometimes on a cumulative basis. The monitoring data from Longwalls 101 to 104 indicates that strains and tilts for general background conditions in an offset geometry are quite different to, and much less than, the strains and tilts observed at stacked goaf edges. Maximum strains and tilts therefore need to be estimated for six different conditions; incremental and cumulative for general background, disturbed or modified ground locations, and at stacked goaf edges or undercut stacked goaf edges.

To estimate maximum strains and tilts, the Holla approach captures the key drivers and allows the differences between the levels of background offset geometries and stacked geometries to be accommodated by varying the constant of proportionality K values.

K values derived from analysis of the subsidence database for Longwalls 101-104 appear suitable to use with the Holla approach to provide a reasonable and conservative estimate of the measured strains and tilts for the six combinations of incremental and cumulative subsidence in general background (disturbed or modified ground) locations, or at stacked goaf edges and/or undercut stacked goaf edges.

Using these K values, estimates for the maximum values for tilts and strains expected at the stacked goaf edge locations can be calculated. The appropriate K values to use at any given location depend on consideration of the direction of mining and how closely the geometry represents a variant of a stacked goaf edge.

The challenge for the proposed ULLD Seam mining subsidence predictions remains in accurately estimating the profile of incremental subsidence and the tilt and strain values from the various components that are now recognised to contribute to the total subsidence movements. Additional factors not significant in single seam mining such as direction of mining relative to existing goafs, separation between existing goafs and latent subsidence effects have a significant influence on estimation of the subsidence profile and maximum tilt and strain levels in a multi-seam operation. Despite these challenges there is substantial confidence that the current predictions contain sufficient conservatism to inform environmental assessment processes.

CONCLUSION

The monitoring data from Ashton allows significant advances in the understanding of multi-seam subsidence behaviour. The characteristics identified indicate that multi-seam subsidence behaviour, although more complex than single seam, is nevertheless regular and reasonably predictable.

The effects of multi-seam mining in modifying the behaviour of overburden strata is highlighted in the magnitude of maximum incremental vertical subsidence as a percentage of the second seam mining height, the magnitude of tilts and strains remote from pillar and goaf edges and in the magnitude of vertical subsidence above the lower seam chain pillars. Other observed effects of multi-seam mining include the difference in behaviour of the overburden strata in response to different mining directions when undermining solid coal/ goaf edges.

The magnitude of vertical subsidence resulting from multi-seam mining has a number of components. A key component in the cumulative subsidence profile is the magnitude and distribution of incremental latent subsidence recovered from the overlying seam.

In areas away from panel and pillar edges, tilt and strain levels from multi-seam mining are likely to be similar to single seam values despite the greater levels of vertical subsidence.

At locations where stacked goaf edges are formed, elevated tilts and strains can be expected. The maximum levels of these parameters is likely to be sensitive to the relative panel geometries in each seam, the dynamics of the mining including mining direction and final mining geometry at these locations.

There is a strong similarity in the characteristics and distribution of horizontal subsidence movements measured above the first four longwalls in the ULD Seam indicating a consistent mechanism driving the horizontal movements and a strong influence of strata dilation in this process.

The magnitude, direction, and form of the horizontal movement for the ULD Seam mining are consistent with horizontal movements observed during mining of the PG Seam longwalls. However subtle differences are seen in the pattern of horizontal movements for the ULD Seam mining, highlighting the influence of the offset geometry and latent subsidence recovered from the PG Seam.

The single seam concepts of angle of draw and subcritical-supercritical width are likely to be less meaningful for multi-seam mining due to the subsequent behaviour of the disturbed (or modified) ground beyond the first episode of mining.

Latent subsidence is a key contributor to the subsidence outcomes. The contribution is seen in all subsidence parameters. Further research is required to better quantify the drivers of the magnitude and extent of latent subsidence effects for each seam particularly in regard to the effect on horizontal subsidence movements. The understanding of latent subsidence effects should also be considered in the design of future subsidence monitoring programs to ensure the maximum subsidence movements are captured.

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EXPERIMENTAL AND NUMERICAL INVESTIGATION OF INJECTION OF COAL WASHERY WASTE INTO LONGWALL GOAF

Zongyi Qin¹ and Baotang Shen

ABSTRACT: Mining, preparation and consumption of coal produce large amounts of waste, of which the coal washery rejects account for a major part. Currently emplacement of coal washery waste not only requires large land, but also pollutes the air, soils and underground water. A number of methods have been developed to make use of coal wastes to fill the voids and strata gaps formed from coal extraction, including dry material backfill, paste backfill and overburden slurry injection. Without the need of underground transport system and interference with coal production, the overburden injection technology is considered a cost-effective method in which the coal washery slurry is injected from the surface down to the caved zone of the longwall goaf and fill the voids. In order to understand the mechanism and behaviour of the grout flowing in the caved zone, laboratory experimental and numerical studies were conducted. The laboratory experiment visually simulated the process of coal washery flowing in the caved zone. The process was also numerically simulated by developing a Computational Fluid Dynamics (CFD) model. These studies provide better understanding of the injection and flow mechanism of the grout in the broken medium. The agreement between the experimental and numerical models indicates that the CFD model is able to simulate the complicated flow and can be used to optimise the injection system design and operation parameters.

INTRODUCTION

Coal production produces a large amount of coal washery rejects. Management of these coal rejects has becomes a challenging issue which affects the mining industry, local communities and governments. Disposal of coal washery rejects is a challenge due to their large volume and the possibility of the waste containing contaminating materials. Currently emplacement of coal washery rejects on the surface not only requires large land, but also causes environmental concerns. The industry needs alternatives to dispose of the large quantity of coal washery rejects. To stimulate improved environmental management of coal washery waste, NSW has introduced the coal washery rejects levy since 2009 with an initial rate of \$15 per tonne. On other hand, extraction of coal underground creates voids including stopes, and goafs and gateroads. These voids can induce instability and subsidence hazards (Sheshpari 2015). Making use of these voids to dispose of coal rejects not only reduces the occupied land and surface pollution but also reduce the subsidence induced by mining.

The voids in the fracture networks of the caved zone of longwall goaf formed by coal mining have the potential to store coal washery rejects. A number of methods have been developed that use coal wastes to fill the voids and strata gaps formed from underground coal extraction, including dry material backfill, paste backfill and overburden slurry injection (Belem and Benzaazoua 2008; Huang *et al.* 2010; Lokhande *et al.* 2005; Mez and Schauenberg 1998). The recently developed Overburden Grout Injection (OGI) technology (Shen *et al* 2010; 2011) is believed to be a more cost-effective method than the dry backfill systems as there is no need for an underground transport system and no interference with coal production.

Overburden grout injection has been successfully used in China to reduce longwall mine subsidence by 40-60% (Shen *et al* 2010). Two previous ACARP projects (Guo *et al* 2005; Shen *et al* 2010) had been carried out to investigate the feasibility of this technology for application in Australian mines. In

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two of the three mines studied in Australia, the OGI technology is expected to reduce subsidence by about 50%. However, before conducting any full scale trial, the practicality of injecting the coal washery rejects into the goaf needs to be investigated. Some key hurdles include interaction between the injection activity and coal production, interaction between gas emission and grout filling, the influence of injection on gas management, and the influence of borehole instability on coal wash injection. In connection with the OGI technology, a concept being considered is to inject the grout directly into the goaf to fill the voids there. This can be considered to be a method mainly for waste disposal but it could also help stabilizing the goaf and reduce surface subsidence. In this case the details of the grout flowing within the caved zone of the goaf and many quantitative parameters about the injection flow rate is appropriate, what range can the injection borehole cover, what space of boreholes should be designed, These unknowns and lack of documented procedures for design and optimisation of injection parameters have hindered coal mine operators from conducting field trials and implement this technology for coal washery rejects treatment.

The paper describes part of the works conducted recently at CSIRO on developing the goaf injection technology, focusing on the laboratory and numerical modelling to visually showing the filling process of the grout in the inaccessible goaf area. Laboratory simulations and Computational Fluid Dynamics (CFD) are combined to investigate the grout injection flow within the longwall goaf. This study consists of three major steps. Firstly a lab scaled transparent injection test system was built up and a series of tests were carried out to visually observe the coal reject grout flowing in the pebble bed which represents the cave zone of the longwall goaf and obtain data for calibration of the CFD model. Secondly, a CFD model was developed based on the lab scaled system to simulate the process of injection. After calibration, the CFD model was further developed to simulate the injection into a large model, based on a real scale of a longwall goaf to understand the process of the coal rejects grout flowing and filling in the longwall goaf.

LABORATORY SCALE INJECTION TEST

It is commonly accepted that a longwall goaf can be divided into three major zones, namely caved zone, fractured zone and separation zone (Shen and Poulsen 2014; Qu *et al* 2015), as illustrated in Figure 1. In this study, a laboratory scaled physical model simulating the caved zone and fractured zone of the longwall goaf was built up, and extensive injection tests was conducted to study the grout flow behaviours within the broken medium using materials provided by the coal industry. The major part of the physical model is a rectangular transparent tank with glass for side walls. The tank dimension is 1200 mm long, 400 mm wide and 400 mm high. On the bottom of the tank a layer of 110 mm of pebble stones are laid. This layer is to simulate the caved zone of the longwall goaf. Snow white pebbles are used for the purpose of easy observation. Above the pebble layer are a number of layers of broken beams made of a mixture of plaster and fly ash, representing the fracture zone of the longwall goaf. The top layer without fracture represents higher overburden strata as shown in Figure 2(d).

Figure 2 shows the apparatus and materials used in this experimental study. The coal washery rejects were provided by South 32 Illawarra Coal. By adding a small amount of water a thick grout was made as shown in Figure 2a. The viscosity of the grout was measured using the Haake Viscometer VT-550 as shown in Figure 2b. The peristaltic hose pump, VF 32, as shown in Figure 2C, with continuously variable speed controller is used to inject the grout into the tank. In running the injection test, an injection flow rate of about 0.035 kg/s was applied by adjusting the speed of the pump. The tank packed with pebbles and brick of plaster and fly ash is shown Figure 2d. A pipe 55 mm in diameter is installed close to one end of the tank. The bottom of the pipe touches the top of the pebble layer. A hose of 15 mm is connected to the pump at one end and the other end of the hose is put inside the pipe



Figure 1: Schematics of structures of longwall goaf overburden strata



Figure 2: Experimental apparatus and materials (a) coal washery grout; (b) viscometer; (c) injection pump; (d) injection tank

The viscosity of the grout varies very much when the coal washery rejects are mixed with different amounts of water. Figure 3 shows the viscosity when the fine coal rejects were mixed with water to reach a density of 1309 kg/m³. It is seen from the viscosity measurement results that the grout behaves like Bingham fluids with viscosity increasing nonlinearly when the shear rate decreases. Figure 4 shows the connected injection system in operation. During the test, it was observed that the grout cannot be seen reaching the front side wall of the tank until one minute passed. As injection continues the grout gradually fills the voids of the pebble layer from one end of the tank towards the other end. After injecting for about eight minutes, the grout in the pipe raised to the top of the pipe and spilled out, marking the final finish of the injection. As the tank sides are transparent the process of grout flow in the pebble layer can be observed and recorded. The data of the position of the flow front at different times was obtained for calibration of the CFD model as will be described below.



Figure 3: Measured relationship of viscosity and shear rate of coal reject grout



Figure 4: Connected laboratory scale injection system

CFD SIMULATION OF THE LAB SCALE INJECTION MODEL

CFD modelling is a useful approach in simulating the detail flow process in the inaccessible areas of the longwall goaf. CFD has been used to investigate the gas flow migration dynamics within longwall goaf areas with the objective of improving gas capture, minimising the risk of spontaneous combustion and developing effective goaf inertisation strategies (Ren and Balusu, 2009; Guo *et al.* 2012). The previous CFD modelling of goaf gas flows have provided better understandings of methane flow patterns in the longwall goaf and helped optimise longwall goaf gas drainage design. In this study, we first develop a CFD model to simulate the coal washery grout flows in the lab scaled injection system. After calibration of the CFD model with the laboratory model, the CFD model is developed further to simulate the real scale model of the longwall goaf to provide information for design and optimisation of the goaf injection system.

Geometry of the lab scale CFD model

The CFD model is based on the lab scaled injection tank. The dimensions are 1200mm x 400 mm x 400mm, exactly the same as the lab tank. A pipe of 55 mm in diameter is located at the right end of the tank to represent the borehole, and a hose of 15 mm in diameter is in the centre of the pipe representing the injection hose. The 3D domain is meshed into 180000 finite cells. The geometry and meshes are shown in Figure 5.



Figure 5: CFD modelling geometry (a) and meshes (b)

Two phase models selected

The Ansys software Fluent 15 is used to develop the CFD model. The injection process is considered time-dependent and the transient flow model was enabled. In addition, it is assumed that the pores of the domain are full of gas prior to grout injection, thus the two phase flow model was used, including air and grout. The volume of fluid (VOF) model was employed to track the interface between the phases. The continuity equation for the volume fraction of the q phase of a two phase model (p and q) is

$$\frac{1}{\rho_q} \left[\frac{\partial}{\partial t} \left(\alpha_q \rho_q \right) + \nabla \cdot \left(\alpha_q \rho_q \overrightarrow{v_q} \right) \right] = S_{\alpha_q} + \sum_{1}^{2} \left(\dot{m_{pq}} - \dot{m_{qp}} \right)$$
(1)

where m_{pq} is the mass transfer from phase q to phase p, and m_{pq} is the mass transfer from phase p to phase q, and S_{α_q} is a source term.

The volume fraction of phase p is computed based on the following constraint:

$$\alpha_p + \alpha_q = 1 \tag{2}$$

The coal washery grout is considered to be a liquid phase whose density of 1309 kg/m³ is measured from the lab test as mentioned previously. The viscosity of the grout was measured and shown in Figure 3. The other phase, air, is considered ideal gas, and the compressible gas law is used to describe the density of the gas.

Porosity and permeability and boundary conditions

The flow domain is a porous medium, and permeability and porosity are used to describe the porous domain. The porosity represents the volume fraction of the flows and the permeability represents the resistance of the medium to the flows. The porosity was determined in the lab through filling water into a certain volume of the packed pebble stones. The permeability is described by introducing the viscous resistance coefficient and the inertial resistance coefficient. To determine these two coefficients, the Ergun equation (Ergun 1952), a semi-empirical correlation applicable over a wide range of Reynolds numbers and for many types of packing are applied:

The viscous coefficient is

$$C_1 = \frac{1}{\alpha} = \frac{150}{D^2} \frac{(1-\varepsilon)^2}{\varepsilon^3}$$

(3)

where

$$\alpha = \frac{D^2}{150} \frac{\varepsilon^3}{(1-\varepsilon)^2} \tag{4}$$

The inertial coefficient C2 is

$$C_2 = \frac{2.5}{D} \frac{(1-\varepsilon)}{\varepsilon^2}$$
(5)

where D is the particle diameter, and ε is the porosity of the packed bed.

The top of the inject hose is defined as mass flow inlet of 0.035 kg/s of grout, as the same as the experiment. Other boundary conditions include boundary walls for the bottom, and the four sides, the top of the pipe and the top of the tank are defined as pressure outlets with atmospheric pressure applied.

CFD simulating results

Figure 6 compares the process of grout filling the voids of the pebble layer of the tank from experimental and CFD modelling. Both are observed from the front side of the tank. The CFD simulation shows that in about 1 minute the grout has reached the front side wall at the bottom right corner of the tank, and gradually moves towards the left. As the filling continues, the interface of the grout and air forms a line with a slope angle of about 40 degrees. This is consistent with the observation of the test. As the permeability of the pebble layer is much larger than the overlying fractured brick layers, almost all of the coal rejects grout flows into the caved zone. Little grout fills in the fractured zone.



Figure 6: Comparisons of injection processes between lab test and CFD simulation

Figure 7 compares the filling distance from the experiment and the CFD modelling, indicating a good agreement between them. It is seen that from both the experiment and CFD modelling that although the injection flow rate keeps constant, the filling speed of the grout is not constant. After 300 seconds, the filling speed gradually slows down. This is because the flow resistance increases as the filling area becomes smaller, and more grout stays in the pipe rather than flows to the voids of the pebble layer.

Figure 8 shows the profiles of the grout fraction (left) and the correspondent pressure profiles at different times on the vertical section crossing the injection pipe and hose. It can be seen that both the grout level in the pipe and the pressure underneath the pipe increase with the injection time. Due to the top end of the pipe (borehole) being open to the atmosphere the driving force for filling is not from the injection hose, but from gravity. It is seen that, as the filling speed slows down, more and more grout is stored in the pipe and the level surface of grout gradually rises up, this in turn builds up pressure and drives the grout flowing further horizontally in the pebble layer. When the grout level in the borehole rises up to the top of the borehole, the pressure underneath the pipe is not able to increase, and the grout is eventually not able to flow further, and the filling process ceases.



Figure 7: Comparison of flow distance of the flow front between test and CFD



Figure 8: Contours of phase (coal washery) fraction (left) and pressure (right) on a vertical section when grout level rises to different positions

CFD SIMULATION OF INJECTION OF COAL REJECT GROUT INTO A LONGWALL GOAF

CFD geometry of the LW3 goaf model

After demonstrating that the developed CFD model is able to simulate the process of the injection of grout into the lab scaled injection model, the CFD model was developed further to simulate a large injection of a real longwall goaf. The longwall goaf is formed by mining a coal seam of thickness of 3 m. The dimensions of the model are 800 m long, 400 m wide and 110 m high. In this CFD model, one borehole located 170 m behind the working face and 50 m from the tailgate is selected as the injection hole. Figure 9 shows the geometry and the meshes of the model of the longwall goaf. The geometry and meshes are shown in Figure 9.

Unlike the lab scaled model where the permeability of the caved zone (pebble layer) can be calculated using the Ergun equation based on the measured porosity and average diameter of the stones, the permeability and porosity of the longwall goaf are not able to be obtained by simple test and calculation. In order to determine the permeability of longwall goaf, a series of site characterisation studies, field investigations, and numerical modelling studies, were previously conducted by researchers CSIRO to investigate longwall mining induced strata behaviours including stress change, strata displacement, and underground water pressure changes in many coal mines of Australia and China. Based on the mine site measurements, 3D models using the software, COSFLOW developed in CSIRO were developed to estimate the permeability changes of the surrounding strata of longwall goaf.



Figure 9: Geometry and meshes for CFD modelling a longwall goaf injection

Permeability of the longwall goaf

These results were used to construct a permeability model of the overlying and underlying strata of the mined coal seam areas for the CFD model. Details of site characterisation and permeability determination for longwall goaf are referred by Guo *et al* (2009; 2012; 2015), and Qin *et al* (2015).

Simulation results of the longwall goaf injection

The simulation of the injection into the longwall goaf can provide information about the grout flowing in the inaccessible caved areas. Different scenarios can be obtained by running this model with different injection parameters, such as injection flow rate, viscosity of the grout and the position of the injection borehole. The filling coverage area, the capacity of disposal of coal rejects in a certain size of goaf, the required injection flow rate and injection time, can be predicted and assessed by running the model. This is particularly useful in design and optimisation of an injection system, in considering the high cost of practising an injection operation.

Figure 10 displays some of the contours of grout fraction from the CFD simulation showing the process of the grout filling the caved zone of the longwall goaf on a longitudinal vertical section, and Figure 11 shows the filled area from the plane view at the working level. It is noted that because the permeability around the perimeter of the goaf is higher than the central area, and the borehole for injection is close to the Tailgate (TG) side, after injecting for 300 minutes, the grout has flowed to the boundary of the tailgate side. If there are no sealing walls between the caved zone and the tailgate, the grout will flow into the tailgate and may influence the mining production. This needs to be taken into consideration when designing and practising a real injection system. By sealing off the cut-throughs and gate roads or locating injection borehole in the central area of the goaf or controlling injection time, grout flowing into gate roads can be avoided.



Figure 10: Coal washery filled area development on a longitudinal vertical section as injection time advances



Figure 11: Coal washery filled area development on a plane section of the working level as injection time increases

CONCLUSION

For the purpose of coal washery disposal in underground coal mines, experimental and CFD simulation of overburden injection of coal washery grout into the caved zone of longwall goaf has been carried out. The lab scaled model uses packed bed of pebbles to simulate the caved zone. The transparent tank allows the filling process of the grout flow in the caved zone to be visible. The developed CFD model successively simulated the grout filling process in the lab scaled injection tank. The calibrated CFD model was then developed to simulate a large model of a real longwall goaf injection. The study not only helps improve our understanding on the mechanisms of grout flow and filling in the networks of the caved zone of longwall goaf, but also provides quantitative flow parameters to optimise the design of the injection system, and assess the capacity of waste disposal in a longwall goaf.

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AN ASSESSMENT OF COAL PILLAR SYSTEM STABILITY CRITERIA BASED ON A MECHANISTIC EVALUATION OF THE INTERACTION BETWEEN COAL PILLARS AND THE OVERBURDEN

Guy Reed¹ and Russell Frith

ABSTRACT: Coal pillar design has historically been based on assigning a design Factor of Safety (FoS) or Stability Factor (SF) to coal pillars according to their estimated strength and the assumed overburden load acting upon them. Acceptable FoS values have been assigned based on past mining experience and at least one methodology includes the determination of a statistical link between FoS and Probability of Failure (PoF).

The role of pillar width: height (w/h) ratio has long been established as having a material influence on both the strength of a coal pillar and also its potential mode of failure. However, there has been significant professional disagreement on using both FoS and w/h ratio as part of a combined pillar system stability criterion as compared to using FoS in isolation. The argument being that as w/h ratio is intrinsic to pillar strength, which in turn is intrinsic to FoS, it makes no sense to include w/h ratio twice in the stability assessment. At face value this logic is sound. However, this paper will argue and attempt to demonstrate that there is a valid technical reason to bring the w/h ratio into system stability criteria (other than its influence on pillar strength), this relating to the post-failure stiffness of the pillar, as has been measured *in situ*, and its interaction with overburden stiffness. By bringing overburden stiffness into pillar system stability considerations, two issues become of direct relevance. The first is the width: depth (W/H) ratio of the panel, in particular whether it is sub-critical or super-critical from a surface subsidence perspective. As a minimum, this directly relates to the accuracy of the pillar loading assumption of full tributary area loading. The second relates to a re-evaluation of pillar FoS based on whether the pillar is in an elastic or non-elastic (i.e. post-yield) state in its as-designed condition, this being relevant to maintaining overburden stiffness at the highest possible level.

The significance of the model being presented is the potential to maximise both reserve recovery and mining efficiencies without any discernible increase in geotechnical risk, particularly in thick seam and higher cover depth mining situations. At a time when mining economics are at best marginal, the ability to remove unnecessary design conservatism without negatively impacting those catastrophic risks that relate to global mine stability, should be of interest to all mine operators and is an important topic for discussion amongst the geotechnical fraternity.

INTRODUCTION

The majority (if not all) of the established coal pillar design methodologies are statistically derived and typically utilise a "classical" pillar strength formulae divided by full tributary area loading (i.e. full cover depth loading) to provide a FoS against core pillar failure. Pillar w/h ratio is typically included as a variable within the pillar strength formulae but otherwise is not formally used to help validate likely pillar stability outcomes as part of a combined system stability criterion. Similarly, potential design parameters such as W/H ratio and/or the presence of thick massive strata units within the overburden (both of which could significantly influence the overburden load acting on individual pillars within a panel) are seldom directly considered.

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The practical consequence of the inability to use these additional parameters when designing mining layouts incorporating load-bearing pillar systems is potentially overly conservative stability outcomes resulting in the unnecessary sterilisation of mining reserves and reduced mining efficiencies.

This paper will demonstrate that there are a number of valid technical reasons to incorporate these factors into the pillar design process via the implementation of a series of logical mechanistic arguments resulting in a more holistic pillar design approach as will now be explained.

COAL PILLAR FAILURE MECHANICS

In order to understand the technical justification for the mechanistic pillar system design approach being proposed, it is necessary to consider briefly coal pillar failure mechanics and the key parameters that are involved.



Figure 1: Illustration of stable and unstable post-failure behaviours

Figure 1 illustrates the well-established concept for stable and unstable behaviour of a structure (a coal pillar system in this instance) once it reaches its ultimate or maximum loading-bearing condition. This includes the two critical elements of (a) the post-failure stiffness of the structure (K_p), and (b) the stiffness of the system that is directly loading the structure (K_M). It is not necessary to explain this in significant detail other than to make the following points:

- (i) It is obviously first necessary for the applied load to exceed the maximum load-bearing ability of the structure in order to drive the system as a whole into a post-failure condition. Without this the structure remains in a pre-failure state and is naturally stable irrespective of the characteristics of the loading system.
- (ii) In the post-failure state, if the stiffness of the loading system (K_M) is less than the post-failure stiffness of the structure (K_p), the system as a whole becomes naturally unstable; as the structure will lose its load-bearing ability at a faster rate than the loading system. As such, whilst ever this condition remains the structure will inevitably progress to a fully collapsed state.
- (iii) Conversely, if the stiffness of the loading system (K_M) is greater than the post-failure stiffness of the structure (K_p), the system will tend to remain naturally stable despite the maximum loadbearing ability of the structure having been exceeded. This is because the structure will lose its load-bearing ability at a slower rate than the loading system hence the system as a whole can attain post-failure equilibrium.

In coal pillar mechanics, the structure is obviously the pillar itself and the loading system is the overburden above it. Therefore, it is necessary to consider both the post-failure stiffness of coal pillars and also overburden stiffness in order to develop a more comprehensive pillar design approach.

Post-failure stiffness of coal pillars has been evaluated by other researchers using both lab-based testing of coal samples (Figure 2 after Das 1986) and *in situ* testing of coal pillars (Figure 3 after Chase *et al*, 1994). These two figures demonstrate the following points, noting that more confidence

is logically placed in the *in situ* test data shown in Figure 3 as it more accurately represents real-life field conditions present in an underground coal mine, as compared to the lab-tested samples shown in Figure 2 and the "filled-in" (i.e. non *in situ*) data points shown in Figure 3:

- (a) Post-failure stiffness decreases as a function of increasing w/h ratio both data sets clearly demonstrate this principle.
- (b) By reference to Figure 3 and the *in situ* test data only, post-failure stiffness becomes "asymptotic" with increasing w/h ratio above about 2. This is in contrast to the post failure stiffness of cases that have w/h ratio values of <2 whereby, post-failure stiffness increases rapidly with ever-decreasing w/h ratio (NB increasing post-failure stiffness is detrimental to coal pillar system stability).
- (c) Post-failure stiffness transitions from negative to positive (which is highly beneficial to system stability) at a w/h ratio, based on an extrapolation of the *in situ* test data in Figure 3, as low as 5.



Figure 2: Stress-strain behaviour of coal for varying width to height (w/h) Ratio (Das, 1986)



Figure 3: Post-failure stiffness of coal pillars as a function of width to height (w/h) ratio (Chase *et al* 1994) – NB open symbols represent *in situ* tests

The data in Figures 2 and 3 allows two other very important statements to be made in relation to the stability and hence design of stable coal pillar systems:

- For w/h ratios of >7 or 8, coal pillars are almost certain to work-harden (or strain-harden) as a
 post-failure behaviour and can therefore be classified as "indestructible" under normal
 overburden loading conditions (i.e. non-bump prone loading conditions) even though they will
 still compress significantly if loaded to a high level.
- 2. For w/h ratios above 2, coal pillar system collapse requires the overburden to have little or no inherent stiffness in order to overcome the potentially re-stabilising influence of the asymptotically low post-failure stiffness of the pillars.

The integrity of these two statements will now be tested in further detail by reference to known failed pillar cases.

AN EVALUATION OF COAL PILLAR FAILED CASES

The previous section of the paper has listed a number of coal pillar system design "rules" by reference to the stress-strain behaviour of coal according to varying w/h ratio. This section will examine those rules by reference to published cases of pillar system failures.

The listed "rules" are evident in the coal pillar failure representation first put forward by Hill (2005) (see Figure 4) whereby:

- (a) the majority (i.e. >50%) of the failed pillar cases included in that database had a design FoS of <1.5 and a pillar w/h ratio <2,
- (b) the density of failed cases starts to reduce for w/h ratios >2 and is effectively almost zero for values >5, and
- (c) the only documented failed case at a w/h ratio of >5 (in the order of 8), which has been the subject of some industry discussion in recent times, has an FoS <1 and was likely to be a floor bearing failure rather than a core pillar failure; this being based on the geotechnical setting, which comprised thick soft floor with a history of allowing remnant coal pillars to punch through (Colwell, 2010).





Case history	State	Depth, m (ft)	Pillar size, m (ft)	ARMPS SF	w/h ratio	Collapsed area, _ha (acres)	Collapse size, m (ft)	Damage from airblast	
Α	ŴV	84 (275)	3 by 12 (10 by 40)	0.86	1.05	2.3 (5.7)	150 by 150 (500 by 500)	26 stoppings, 1 injury.	
B1	WV	73 (240)	3 by 12 (10 by 40)	0.96	1.00	-	-	32 stoppings, fan wall out.	
			3 by 18 (10 by 60)	1.10	1.00				
B2	WV	75 (245)	3 by 12 (10 by 40)	0.94	1.00	1.7 (4.1)	100 by 150 (350 by 500)	40 stoppings.	
B3	WV	85 (280)	9 by 9 (30 by 30)	1.46	3.00	2.8 (6.8)	180 by 180 (600 by 600)	70 stoppings.	
			6 by 12 (20 by 40)	1.47	2.00				
C1	wv	60 (195)	3 by 12 (10 by 40)	1.19	1.00	2,1 (5.2)	140 by 150 (450 by 500)	103 stoppings.	
C2	wv	99 (325)	9 by 9 (30 by 30)	1.15	3.00	1.9 (4.8)	100 by 180 (350 by 600)	Minimal.	
D	wv	69 (225)	6 by 6 (20 by 20)	1.15	1.82	1.7 (4.3)	100 by 160 (350 by 540)	37 stoppings.	
			9 by 9 (30 by 30)	1.42	2.73				
E1	wv	91 (300)	3 by 12 (10 by 40)	0.79	1.42	7.4 (18.2)	240 by 290 (800 by 950)	Major damage,	
E2	wv	91 (300)	3 by 12 (10 by 40)	0.71	1.11	6.7 (16.6)	220 by 275 (720 by 900)	Major damage.	
F	ОН	76 (250)	2 by 12 (7 by 39)	0.66	2.12	2.0 (4.9)	90 by 215 (300 by 700)	Minimal.	
G	UT	168 (550)	12 by 12 (40 by 40)	0.95	2.29	7.9 (19.4)	150 by 490 (480 by 1,620)	Major damage, 1 injury.	
o	wv	_		1.03	2.50	1.8 (4.5)	120 by 150 (400 by 500)	_	
R	co	120 (400)	4 by 24 (12 by 80)	0.57	1,71	2.8 (6.8)	180 by 150 (600 by 500)	Minor damage.	
NOTE Dash indic	ates no d	lata available.							

Table 1: Massive pillar collapses in US coal mines (Mark et al 1997)

The failed cases data in Figure 4 is also mirrored in the US failed cases described by Mark *et al* (1997) (see Table 1) and summarised in Figure 5. In this regard, it is noted that ten out of the sixteen failed cases have a w/h ratio of ≤ 2 (with none being >3) while all SF values are <1.5. Again, the substantial stabilising effect of combining a design FoS of at least 1.5 with a pillar w/h ratio no less than 3 to 5 is clearly evident.



Figure 5: ARMPS SF v pillar w/h ratio for pillar collapses and other case histories (NIOSH 2013)

What this all leads to, is a potential resolution to the arguments and disagreements that have arisen due to the original publication of Figure 4. Galvin (2006) made the point in relation to the representation in Figure 4, that pillar w/h ratio was included in both axes; as it was already part of the FoS calculation through its inclusion in pillar strength formulae. This is absolutely correct and at face value appears to justify that this type of graphical representation of failed cases has no merit and could in fact be misleading. However, if it is accepted that pillar w/h ratio also has a significant influence on post-failure pillar stiffness, and this has a controlling influence on whether a coal pillar collapse will occur or not, then the Hill (2005) representation has significant merit. The argument that w/h ratio is included in both axes of the graph is not a valid reason to dispense with the representation.

The other coal pillar system design "rule" emanating from Figure 4 relates to pillars with w/h ratios <2 and their seeming ability to be prone to failure/collapse at FoS values that should otherwise not occur. The commonly stated reason for this is that at such low w/h values, coal pillar strength can be significantly compromised by the presence of localised geological structures, such as joint swarms,

faults, dykes etc. as compared to higher w/h ratios whereby a confined pillar core is likely to be developed irrespective of the weakening defects within the pillar. This issue simply dictates that other pillar system stability controls need to be put in place when developing a panel or mine layout incorporating large numbers of coal pillars with w/h ratios of <2 as will now be described in relation to using the stiffness of the overburden as a pillar stability control.

THE ROLE OF OVERBURDEN STIFFNESS

Having detailed the influence of both pillar FoS and w/h ratio as independent parameters influencing the role of the coal pillar in pillar system failures, it is now necessary to address the role of the overburden. Based on Figure 1 it is evident that the post-failure stiffness of the overburden needs to be suitably low for coal pillars to be driven to a state of full collapse once they have been over-loaded (as described previously).

An instructive way to address overburden stiffness is to use the established concepts of "sub-critical", "critical transition" and "super-critical" surface subsidence as illustrated in Figure 6 with actual subsidence data being provided in Figure 7 (this representation being known colloquially in Australia as a "Holla" curve after the late Lax Holla).

The point of this is to demonstrate that it is only in the super-critical range, whereby the entire overburden to surface loses most (if not all) of its inherent stiffness so that it effectively then behaves as a "detached" loading block (with no inherent stiffness), that can drive over-loaded coal pillars to a full state of collapse. Conversely, in the sub-critical range, at least a portion of the upper overburden is demonstrably being controlled by either the excavation geometry or the spanning capabilities of massive strata units (or both), which by definition must therefore retain some level of stiffness within part of the overburden in that its natural settlement at surface under gravity is being restricted.

Evidence for the controlling influence of W/H ratio on coal pillar system failures can be found in Table 1 and also the un-published results of a study into pillar failures in highwall mining where large numbers of coal pillars with very low w/h ratios are commonly used. The US data presented in Table 1 contains minimum W/H ratio values of >0.9 but typically >1.5 for all collapsed cases with the unpublished highwall mining collapsed cases again being exclusively associated with W/H ratio values >0.9. It is noted that failed cases information published by the University of New South Wales (UNSW) are insufficiently detailed to allow this same analysis.



Figure 6: Schematic representation of the mechanics of sub-critical ("deep" beam) and supercritical ("shallow" beam) subsidence behaviour (Ditton and Frith, 2003)



Figure 7: Measured S_{max} values analysed according to extraction height (T), panel width (W) and cover depth (H) (Ditton and Frith, 2003)

The significance of a W/H value in the order of ≥ 0.9 is immediately obvious in Figure 8, which contains measured surface subsidence data (S_{max}) for cover depths in the range 70 m to 150 m. The red dotted line represents the "mid-point" of the critical transition, whereby values of W/H >0.8 tend towards being super-critical but values <0.8 tend towards being sub-critical. The point is that a minimum W/H value of 0.9 has been found in two separate studies on two different continents as being the lower defining value for failed pillar cases. This strongly confirms (a) the important role of super-critical overburden behaviour and hence low overburden stiffness to surface in pillar collapses and just as importantly, (b) the potential additional stabilising influence of W/H values <0.8 when coal pillars have been designed for full tributary area loading.

Following on from the description of the influence of W/H ratio on overburden stiffness to surface according to different surface subsidence conditions, the influence of lithology on overburden stiffness for a given panel width will now be considered.

Two fundamental studies will be referred to in this regard, one relating to the influence of thick nearseam massive strata units on overburden periodic weighting and caveability as it effects longwall face stability (Frith and McKavanagh, 2000) and the other related to the ability of massive strata units to influence surface subsidence magnitudes (Ditton and Frith, 2003).

Without digressing into significant technical detail, the periodic weighting classification developed by Frith and McKavanagh 2000 (see Figure 9) provides a useful first approximation as to how a massive strata unit may behave (i.e. collapse or span an opening) based on its thickness, the extraction panel width and its material type (specifically conglomerate or sandstone). The defined *"bridging shortwall"* outcome is likely to result in overburden spanning and therefore, inevitably a reduction in surface subsidence due to overburden sag from which, the retention of significant overburden stiffness can be reliably inferred.



Figure 8: Measured S_{max} values analysed according to extraction height (T), panel width (W) and cover depth (H) for depths ranging from 70 m to 150 m (Ditton and Frith, 2003)



Figure 9: Periodic weighting classification (Frith and McKavanagh, 2000)

The potential spanning phenomenon associated with thick and massive strata units in the overburden was also recognised and defined by Ditton and Frith (2003) in relation to the ability of certain strata units to reduce levels of surface subsidence over and above what W/H ratio alone would suggest. Figure 10 is provided as a reference source relating to what is termed as *"Subsidence Reduction Potential"* (or SRP).



Figure 10: Subsidence reduction potential (SRP) according to strata unit thickness, location of strata unit above the seam and panel width (Ditton and Frith, 2003)

As an example, for a panel width of 120 m, the strata unit thickness above which spanning of that unit can be reliably inferred is just <20 m (marked as red circles in Figures 9 and 10). In other words, two different classification schemes that were developed to address different mining outcomes show a very close correlation in terms of the onset of strata unit spanning across an extraction panel of given width.

Figure 10 allows the analysis to be taken a stage further as it brings in the varying location of a thick massive unit within the overburden, the higher the unit above the extraction horizon (as given by y/h in Figure 10), the lower the unit thickness required to develop high Subsidence Reduction Potential (SRP). This makes sense when natural arching and consequent narrowing of the span above an extraction panel due to caving is considered (refer Figure 6 for an illustration of this concept). At a distance of half the cover depth above the extraction horizon (i.e. y/h = 0.5), the unit thickness required to modify surface subsidence across a 120 m wide panel is only 50% of that required when the unit is present in the immediate roof (i.e. y/h = 0).

Therefore, with knowledge of the W/H ratio of a proposed panel of pillars combined with the thickness and location of significant lithological units within the overburden, it is possible to make credible predictions of whether coal pillars will be loaded under full tributary area loading to surface by a "soft" loading system (as is commonly assumed in pillar design) or whether the overburden has the ability re-distribute overburden load to adjacent barrier pillars or solid coal due to its inherent stiffness. This is a useful layout aspect to bring into the pillar design process and further develops the design criterion contained within ARMPS-HWM whereby the number of HWM plunges between barriers is limited to 20.

OVERBURDEN LOAD DISTRIBUTIONS WITHIN A PILLAR SYSTEM

If one uses the concept of sub-critical panel width between barrier pillars (or solid abutments) in coal pillar design, the concept of coal pillar FoS is modified to coal pillar system FoS. In practical terms what this means is that the stability of any smaller coal pillars between the larger barrier pillars needs to be evaluated with the barrier pillars also included within the overall pillar system. This changes the definition of a barrier pillar from one that has the ability to truncate a coal pillar run, to one that has the ability to prevent the pillar run in the first instance.



Figure 11: Bord and pillar type assessment of pillar stability (pillar load distribution based solely on individual pillar width)



Figure 12: "Double goaf loading" of pillars within a sub-critical panel bound by suitably sized barrier pillars of solid abutments (worst case unequal pillar load distribution)

Figure 11 contains an illustration of a coal pillar system containing small pillars located between larger barrier pillars and illustrates the basic scenario of individual pillar loading being based solely on individual pillar width. This allows individual pillar FoS values under full tributary area loading to be determined, along with an overall system FoS for the combined influence of both the small pillars and the barriers.

To demonstrate how one may evaluate the potential influence of overburden load re-distribution due to the sub-critical nature of the spans between barriers, Figure 12 presents the same sub-critical panel layout of small pillars with the initial load exceeding their strength. Due to the sub-critical nature of the panel, overburden load is re-distributed to the adjacent larger barrier pillars. The worst-case example of this is found by assuming that an extraction goaf or gob has effectively formed between the adjacent panel barriers (or solid abutments) so that:

- a. the overburden load acting on the barrier pillars increases, but
- b. the overburden load acting on the smaller in-panel pillars consequently decreases

It is not being suggested that such a situation, including the necessary significant overburden fracturing via the development of a caving angle, can realistically develop within such a layout. It is simply one method of demonstrating that for sub-critical panel geometries, it is seemingly mechanistically improbable for the overburden to drive low FoS pillars between larger barriers to failure, the panel geometries of known failed cases supporting this assertion

COMMENTS ON DESIGN FACTOR OF SAFETY

The current use of pillar FoS or SF is based largely on a statistical assessment of failed cases, the idea being to ensure that the design value used is sufficiently conservative so that the various unknowns or vagaries of the design problem do not in practice, combine to cause a pillar system failure whereas the analysis indicated otherwise. As a basis for further discussion, this paper suggests another possible interpretation of Factor of Safety based on the concepts presented herein, which are all based around the interplay between coal pillar stiffness and overburden stiffness rather than simply pillar strength/load.

With the exception of the failed HWM cases in Figure 4, all of the collapsed cases in both Figure 4 and Figure 5 are associated with FoS or SF values <1.5. There are no collapsed cases above this, yet the UNSW Pillar Design Procedure (PDP) extrapolates beyond this to determine Probability of Failure (PoF) values for FoS values that are well above 1.5.

The question being raised in this paper is whether there is in fact a mechanistic reason as to why the collapsed cases truncate at a maximum FoS of around 1.5, such that there is then perhaps a reason to argue that for values >1.5 the potential for pillar collapse is effectively eliminated for mechanistic reasons. If this were shown to be the case, it would necessitate a complete re-consideration of the statistical evaluation of failed cases for design FoS guidance above 1.5. The practical significance of such a change in approach would be quite considerable.

If one accepts that a specific role of coal pillars is to limit overburden movements to maximise the level of overburden stiffness that is retained (thus assisting overall system stability), then a different interpretation of FoS in the failed cases is forthcoming. If one assumes that the strength formula provides for a reasonable approximation of the maximum loading-bearing capacity of the pillar, a design FoS of 1.5 would approximately represent the pillar being loaded at or close to its elastic limit (i.e. Hooke's Point). For FoS values above 1.5, the pillar would be in an elastic state, whereas below 1.5 it would enter a non-elastic state with an ever-decreasing stiffness towards its maximum strength.

In other words, for FoS values above 1.5, the coal pillar is most likely to remain in an elastic state whereas for values below it is far less likely. In terms of overburden stiffness being maximised by minimising overburden settlements, the difference between a FoS of 1.4 as compared to 1.6 would be highly significant when considered in this manner.

The work has not been done to prove this hypothesis. However it is interesting to consider that there may be a mechanistic explanation for collapsed cases almost always having pillar FoS values <1.5, rather than simply assuming that it is all based on design uncertainty and therefore applying statistical methods to address the problem of determining acceptable design FoS values to prevent future collapses.

SUMMARY

This paper has outlined various technical arguments for the use of a mechanistic and far more holistic approach to coal pillar system design, whereby the independent influences of w/h ratio, W/H ratio and the presence (or absence) of thick massive strata units within the overburden are considered in conjunction with pillar FoS. The objective of combining these various parameters is to provide far more robust design outcomes where more than just the strength of the coal pillar is acting to promote system stability. The potential mining advantage of doing this is in being able to design more efficient mining layouts that recover more of the available coal reserves.

The ability to combine the stabilising influences of occasional high w/h pillars within a mining layout and sub-critical working panels according to both geometry (W/H) and/or spanning strata units within the overburden, may allow for the development of stable mining layouts that would have previously been discarded on the basis of the smaller production pillars within the system having insufficient FoS or SF under full tributary area loading. This is of particular relevance to thick seam bord and pillar workings in deeper cover whereby mine design utilising only FoS under full tributary area loading is highly restrictive.

In a more general sense, shifting the focus of coal pillar design from a simple load balance to one of maximising the stiffness of the pillar system and the consequent minimisation of overburden movements as an aid to global stability, is analogous to the change from roof suspension to roof reinforcement that transformed the way that mine roadway roofs are stabilised with rock bolts. This is in an intriguing possibility to consider and one that will be the subject of a future research.

At a time when mining economics are at best marginal, the ability to remove unnecessary design conservatism without negatively impacting those catastrophic risks that relate to global mine stability, should be of interest to all mine operators and are an important topic for discussion amongst the geotechnical fraternity.

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HUME COAL – AN OVERVIEW

Ben Fitzsimmons and Rod Doyle¹

ABSTRACT: Hume Coal is a subsidiary of POSCO, the South Korean steelmaker. It maintains a 100% interest in the Coal Exploration Authorisation 349 (A349) in New South Wales near Sutton Forest. This paper describes the exploration activities undertaken to date both historic and more recent as well as how the results of those activities have assisted in developing the proposed mine plan. Land Access has proved to be difficult in places. Located near the south western edge of the Sydney Basin the stratigraphy is different to the normal sequence towards the centre of the basin.

Mining Lease Applications have been submitted to the NSW government and an Environmental Impact Statement (EIS) has been prepared and will be on Public Display early in 2017. Mining constraints and their impacts on the mine plan are also described.

INTRODUCTION

The paper summaries the current status of the exploration results to date in the Authorisation 349 - (A349), an exploration tenement in the Southern Highlands of New South Wales. Located near Sutton Forest and the Belanglo State Forest, A349 is situated astride the Hume Highway. An EIS has been prepared and describes the proposed underground mine. The proposed mining technique, mining area and other aspects e.g. hydrology, as well as the current status of the project, is discussed.

The Southern Highlands has been explored for generations and has had several underground operations, generally all small scale by today's modern standards. Nevertheless mines such as Erith Colliery within the Morton National Park represent a long and historic connection to coal mining in the region. The recently closed Berrima Mine is some 5 km kilometres to the NNW of A349.

In A349 there are some 164 historic holes and Hume Coal in the last five years has drilled some 146 drill sites, including, large diameter holes, slimline exploration holes, piezometers and open holes. Total exploration holes drilled in the area equals 346. Several remote exploration techniques have also been used including; seismic, aeromag and ground magnetometer surveys.

Geological structures located in the area have been identified from, surface mapping, from remote techniques and from exploration drilling. Some structures have been strongly defined with very high confidence and some structures are hypothesised and of much lower confidence. The landscape varies between flat lying to incised gorges in the area where the Hawkesbury Sandstone is present, to undulating grazing countryside where the Wianamatta Shales are located.

The target coal seam is the Wongawilli seam and the seam varies from being not present (eroded from the sequence) to its normal (i.e. coastal) 9m thickness. A unique aspect about the deposit is the removal or lack of deposition of the Narrabeen sequence which is not present on the tenement. In most places the Hawkesbury Sandstone has eroded into the Wongawilli seam. In addition many holes have intersected the American Creek and Tongarra seams.

LOCATION

Located in the southern portion of the Sydney Basin, A349 currently occupies some 89 square kilometres. Figure 1 highlights the area and the Mining Lease Applications. Interestingly, there are predominantly two types of land use in the area and these are more or less based on geology. The two main rock formations are the Hawkesbury Sandstone to the west and the Wianamatta Group, predominantly shales to the east of the Hume Highway. The Wianamatta Groups is represented by

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extensive (and expensive), predominantly cleared grazing land. The area of the Authorisation still has some minor pockets of the original flora which have not been completely cleared, i.e. the Southern Highlands Shale Woodlands. While the Hawkesbury Sandstone dominates the Belanglo State Forest an agricultural enterprise growing radiator pine trees on the sandy soils, but more infamous for the tragic murders that took place therein. An area managed by the State Forestry Department.

The landscape is generally speaking undulating countryside with a couple of creeks that have incised the area in the western fringe of the Authorisation. Some dominate igneous structures adorn the region with Mount Gingenbullen, in the north-east of A349. Joplin (1964) identified this structure as an 'unroofed sill'. Other igneous activity has also had its impact on the environment with diatremes being identified by aero and ground magnetometer surveys and also exposed by cuttings of the Hume Highway. Basalt flows also form distinct exposed landforms on some of the properties in the area.



Figure 1: Location map of A349 and MLA527, 528 and 529 over a grey scale elevation grid

STRATIGRAPHY

The stratigraphy is depicted in Figure 2 below. The most distinctive thing to note is that there is no Narrabeen Group to be found in the area of the Authorisation. Indeed there are no upper units of the Illawarra Coal Measures present either. Over the authorisation an angular unconformity exists near the contact between the Hawkesbury Sandstone and the Wongawilli seam. However in places the seam has been completely eroded away by the erosive sandstone. There is some uncertainty associated with the nature of this scenario as the edge of the Sydney Basin is quite close. Because of the proximity to the edge of the basin there is a possibility that portion of the Upper Illawarra Coal

Measures and all of the Narrabeen Group were not deposited in this area, but developed deeper into the central part of the basin (to the east). Also given the deterioration of the Bulli seam in the south of the Illawarra (as noted in Harper, 1915) this lack of deposition has some merit.

In the, 'Explanatory Notes for the Moss Vale 1:100,000 Geology Sheet 8928', Trigg and Campbell (2016) note that there is only minor Narrabeen's which are present well to the south east of A349.

SYDNEY BASIN STRATIGRAPHY						Thick	Avg Ash	Sec	tion
TYPICAL STRATIGRAPHY			A349 STRATIGRAPHY	Y		-	-	Unite	eroded
		Wianamatta Group	Wianamatta Group	R		0.35	-		
		Hawkesbury Sandstone	Hawkesbury Sandstone						
TRIASSIC	Чſ	Newport Formation	Absent		FCM	?	-	Only present in the North of A349	
	SOL	Bald Hill Claystone	Absent		I CIVI				
	I GI	Bulgo Sandstone	Absent	-					
	EEN	Stanwell Park Claystone	Absent		Α	0.47	39.3		
	βAB	Scarborough Sandstone	Absent	-					
	ARF	Wombarra Claystone	Absent		В	0.50	33.1		
	Z	Coalcliff Sandstone	Absent	Ī	<u> </u>		<i>c</i> 2 <i>i</i>		
PERMIAN ILLAWARRA COAL MEASURES		Bulli Coal	Absent		C	0.47	63.4 35.7		
		Loddon Sandstone	Absent		D				
		Balgownie Coal	algownie Coal Absent						
	S	Lawrence Sandstone	Absent						
	URE	Cape Horn Coal Member Absent				0.91	32.1		
	EAS	Hargrave Coal Member	Absent	_					
	ME	Woronora Coal Member	Absent		F	0.37	33.7		
	JAL	Wongawilli Coal	Wongawilli Coal	-		0.77	31.2		
	A CC	Kembla Sandstone	Kembla Sandstone		G				
	RR/	merican Creek Coal Member American Creek Coal Member							
	٨A	Darkes Forest Sandstone	Darkes Forest Sandstone		н	0.34	35.0		
	IA.	Tongarra Coal	Tongarra Coal	-		1.23	21.1		
		Wilton Formation	Wilton Formation						
		Erins Vale Formation	Only a few older (less roliable)	Į	Т				
		Pheasants Nest Formation	holes have drilled below the						
		Shoalhaven Group							
		Talaterang Group	buse of the rongaria Coal.		J	0.35	41.0	Where Pr	resent

Figure 2: Stratigraphic table of the general Sydney Basin compared to A349 and WWSM detail.

In the A349 area there are only three coal seams that have been recognised, the Wongawilli, the American Creek and the Tongarra. The Wongawilli is the only economic coal seam. It ranges in thickness from 0.44 to 8.56 m, and averages 4.77 m. The Wongawilli seam will produce a 10% ash coking coal with good coking characteristics and a middlings thermal product. It is expected that a 3.50 m thick mining section will be extracted. The American Creek is too thin to be economic and ranges from 0.04 to 1.13 m in thickness and frequently splits into two parts. While the Tongarra ranges in thickness from 0.81 to 6.78 m averaging 3.64 m and is also split in places. Laboratory testing of ten Tongarra seam intersections averages 56% raw ash with a CSN of up to 1.5.

EXPLORATION ACTIVITIES AND RESULTS

There are 167 historic holes drilled in the A349 area. These holes were drilled in the 1970's. In addition Hume Coal has drilled 179 exploration and water piezometer holes. This combines to a total of 346 holes, in an area of about 89 square km. On average this represents about 4 holes per square kilometre or holes with a radius of influence of about 300 m, in other words a reasonably well drilled

out resource on a well-known coal seam. However, some areas have a lack of holes and others are drilled at closer spacing than the above. This tends to reflect the difficulty in obtaining land access with current landowners.

As a general rule the Wongawilli Coal is thicker in the north of A349. While in the south and in particular the south-west of the authorisation the seam is subject to significant erosion and in some locations the seam has been eroded away altogether.

The Wongawilli Coal dips gently from the west to the east with 100 m elevation change over 10 km, a grade of 1 in 100. The seam outcrops in the west in Longacre Creek where there is an adit at about 50 m below the top of the Hawkesbury Sandstone cliffs at this location. While in the south east the seam is about 200m below the surface.

GEOLOGICAL STRUCTURES

There are numerous geological structures that have been identified in the region. The Mount Gingenbullen dolerite is the most dominant igneous feature within A349. A small quarry was mined on the mountain's north eastern flank. Further to the north near Bowral, Mt Gibraltar a micro-syenite intrusion occurs, while Mt Jellore similarly is a micro-syenite intrusion and is situated to the west of Bowral. There are also numerous Tertiary igneous flows (remnants of the Robertson Basalt) in the area. Diatremes are also well known in the area. Aerial magnetics were flown over A349 by Shell in 2002, this work was done at a scale that picked up numerous anomalies, but unfortunately was not able to identify dykes. Drilling targeted several diatremes which were proved up by drilling and surface mapping as a result.

Surface magnetics was undertaken on several properties over the authorisation during 2014 and 2015, this work provided significantly enhanced images, but again the surveys did not readily locate igneous dykes. Suggesting the dykes are either very thin or are essentially low in magnetic content. Figure 3 compares the same area with the 2002 aeromag survey image to the more recent 2015 ground magnetometer image. Borehole evidence has identified numerous igneous features. The scale (eg throw or thickness) of these features are largely open to interpretation.



Figure 3: The 2002 aerial magnetics on LHS compared with 2015 surface magnetics on RHS

Using Minex software a geological model was initially developed, through which cross sections were sliced to determine the general nature of grades, points of inflexion or potential faulting within the Wongawilli Seam. In addition a review of all borehole logs was undertaken, which examined both historic and Hume Coal holes. This review also studied every core photograph to interpret possible structures. The process indicated a broad range of geological structures ranging from joint zones of diverse intensity to faulting and igneous intrusions. The interpretations were then compared to the remote sensing results such as seismic and magnetic data, as well as aerial and surface mapping

work. Geological structures were interpreted and assigned different levels of confidence. The interpreted structures were then used to assist with designing the proposed mine plan. The result is a Mine Plan that recognises the potential for geological impacts.

LAND ACCESS

Following the government requirements for exploration activity and in an attempt to fulfil its licence conditions Hume Coal sought to undertake its legal exploration activities e.g. continued exploration drilling, geophysical logging and surface magnetic surveys. A Review of Environmental Factors (REF) was provided to the government for their approval. Following the REF being processed an opposition group took the Minister of Mines and Hume Coal to the Land and Environment Court (LEC) to appeal the approval. The decision was made in favour of the Minister and Hume Coal, citing that the exploration activities would not cause significant impact.

Under the approved REF three boreholes were drilled twoon private property and one on Hume Coal controlled land). Several landowners took Hume Coal to the LEC to appeal against already arbitrated land access, based upon the interpretation of 'significant improvements'. The LEC hearing initially found in favour of Hume Coal, however this decision was appealed and the decision was found to have erred in law so the original judgement was set aside. Since then the NSW government have revised the legislation in an attempt to clarify what 'significant improvements' were made (NSW Government revised legislation). Difficulties in obtaining land access has directly resulted in not being able to further improve the level of confidence in the Resource Assessment from Inferred to Indicated or to a Measured status.

WATER CONSTRAINTS UPON MINE DESIGN

The Hawkesbury Sandstone Formation a well known for its ground water systems and supplies water to many of the local farms. Some water bores are likely to be associated with joint zones or other geological structures. In an attempt to minimise harm to the ground water systems and to surface structures a decision to reduce the resource recovery and not to undertake longwall or shortwall mining, nor bord and pillar mining or any secondary extraction that might cause a goaf to be developed, was made. As a result an effective 'first workings' mine plan was devised with pillars left to ensure that subsidence impacts would be negligible.

This design will essentially retain the current levels of permeability within the strata, which will reduce the inflow of water into the underground workings. Had a longwall or similar goaf forming operation been designed greater inflows into the mine would have been experienced. To regulate water inflows into the mine, the older areas of the mine will be systematically sealed off from the active areas. The seals will be substantial enough to maintain the head of water and as such any impact upon the ground water table levels. As a result regeneration of the water table will be faster.

In the region surrounding the Hume Coal Mine, there are four main rock types. Starting from the top and working down – they are:

- Igneous rocks, flows such as the Robertson Basalt. In general, these rocks have reasonable groundwater.
- Rocks from the Wianamatta Group, which contain a mix of shales and sandstone. These rocks were originally deposited off shore in the ocean and hence have a high salinity, the water is unsuitable for most human and agricultural needs.
- Underlying this is the major sandstone formation known as the Hawkesbury Sandstone. This sandstone has significant fresh groundwater resources, although minerals such as iron and calcium impact on its quality.

• In the A349 area the Hawkesbury Sandstone lies immediately above the Wongawilli seam. The Wongawilli seam also contains groundwater of similar quality to the water in the Hawkesbury Sandstone.

There are numerous water bores in the area of the proposed Hume Coal Mine. Many people pump water from the Hawkesbury Sandstone and use it for both domestic and agricultural purposes, a few people also use the water for irrigation. Mining at Hume will have variable impacts on water bores within the mining area and to a much lesser extent outside the area of mining. The scale of impact will differ depending on where the bores are located relative to the mine. Hume Coal will develop a 'Make Good' programme to ensure that no landowner will be worse off with regard to groundwater as a result of mining.

Extensive water samples from numerous bores and surface sites have been gathered by Hume Coal over several years to develop an understanding of the 'baseline' groundwater conditions. In addition the levels of water in the various strata horizons have also been measured. This data has been used to develop a significant and sophisticated computer model of the groundwater behaviour and how it will react to Hume Coal's proposed mining.

In addition several specialists were engaged to determine how the interaction of mining and groundwater would be affected and how any impacts would be mitigated. One such study (RGS – Geochemical Assessment) involved considering the chemistry of the ground water and how it would be impacted at every stage of mining. These stages included;

- Raw and washed coal stockpiles will have short term exposure for various parcels of coal, water from these stockpiles will be collected and monitored.
- Surface drift stone and surface reject stockpiles will be covered with soil and grass. Water
 will be kept away from these areas although some rainfall may seep into the stockpile. Any
 run off will be collected, monitored and treated as required. Drift spoil materials (Hawkesbury
 Sandstone) will be pH neutral and are considered to be non-acid forming. They have a low
 sodicity and therefore have a low risk of dispersion or erosion. That is, they will remain intact
 on the surface until they are returned for backfilling in the underground environment.
- Rainfall remote from stockpiles etc will be diverted away from mine collection.
- Underground coal reject emplacement will be treated with 1% calcium carbonate (CaCO3) to neutralise the water. CaCO3 is an alkaline material which will offset the acidic nature of the rejects and bring it back into balance.
- Overall any water coming into contact with coal will not be released from site, without being treated in the Water Treatment Plant (WTP).
- Laboratory studies were undertaken on representative rock samples from boreholes. This included where;
- the drifts and shafts are planned to be mined,
- the coal being mined,
- the coal left unmined in-situ, and
- reject material (bands from the coal seam non product) returned to the underground voids.

It's worth noting that despite the testing already undertaken, on-going monitoring will take place throughout the life of the mine. This will ensure that the ground and surface water systems will remain neutral, ensuring Hume Coal's aim of environmental sustainability.

MINE PLAN

Hume Coal is proposing an underground coal mine which will use a combination of traditional first workings drivages and an adaptation of high wall remote continuous mining techniques. The main driver for the design of the Hume Coal Project is the environmental constraints associated with the

project setting. Hume Coal recognises the importance of the groundwater resource, and minimising the impact upon it by mining. Therefore, it was crucial to ensure the mine design allow for the safe extraction of coal while protecting the overlying strata and groundwater systems.

The underground mining operations will be accessed by two drifts, one of which will allow the entry and exit for personnel and equipment, and the second for the clearance of mined coal. Shafts will also be constructed to assist with the ventilation of the mine.

Main roadways and gate roads will be developed using continuous miners of conventional dimensions. The plunges to be developed sub-perpendicular to the gate roads on both the left and right sides are known as plunges. The plunges will be made with an approximate cross sectional area of 4 m wide by 3.5 m high by using remote control continuous miners. The depth of the plunge will vary throughout the mine; however they will extend on average, 120 m.

Each plunge will be separated by a long and slender coal pillar which will ensure adequate stability for the overlying strata, eliminating fracturing of the overlying rock mass and consequently limiting any long-term impacts on overlying groundwater systems and surface infrastructure. Plunges will be grouped into 'panels' which will be roughly 60 m wide and consist of approximately seven plunges and six long pillars. Each panel will be separated by a barrier pillar of approximately 16 m width. Plunges will be developed from the end of the gate road, retreating back toward the mains roadways. Coal will be cleared from the plunge using technology similar to Joy's floor mounted Flexible Conveyor Train (FCT) back to the gate road. Once completed, bulkhead seals will be installed to seal off the mined-out area.

Exploration work has identified that there is little gas present in the authorisation. An average level of 0.3 m^3 /t is present, of which some 95% is Carbon Dioxide with the remainder being methane. As such the level of gas underground is expected to be minimal.

Hume Coal will dispose of rejects into the underground mined out workings. The volume of rejects will equate to approximately 20% of all Run of Mine (ROM) production. Aside from the advantage of not requiring an extensive surface area to dispose of the reject materials, the mined-out areas will be partially filled, inbye of the bulkhead seals. This will allow for a more rapid recovery of groundwater levels as mining progresses.

Rejects which are produced through the processing of the coal will be transported to a reject material treatment plant which will crush the material and mix with water, resulting in a material similar to the consistency of toothpaste. Calcium Carbonate will also be mixed with the rejects to assist bringing the water into a neutral balance. The reject material will then be pumped underground for permanent storage.

The mine will aim to produce 3.5 million tonnes per annum (Mtpa) ROM, with a saleable product of 2.8 Mtpa. The ROM coal will be processed on site and produce two products, metallurgical coal with an average ash of 10% and industrial coal with an average ash of 22%. Processing of the coal will consist of crushing, gravity separation and floatation. The product coal will be stockpiled on site, separated into the two different products. Coal will be transported to Port Kembla using rail. The rail wagons will use covered coal wagons, which will ensure best practice techniques in the management of air quality.

It is because of Hume Coal's desire to minimise the impact on the ground water that the mine is designed as a "Low Impact", a first workings only underground coal mine. As a result of this there will be no goafing, and therefore no ground stability issues with surface subsidence being predicted to be less than 20 mm.

Figure 4 highlights the mine plan following numerous revisions and elimination of any goaf formation. There are several limitations on the nature of the mine plan, including; the thickness of the working



section, geological washouts and structures as well as heritage properties (adjacent to Golden Vale Road), a dam notification zone and some surface infrastructure.

Figure 4: Hume Coal's proposed mine plan, showing the mining lease application areas in green outline. Note MLA527 and MLA528 – underground and MLA529 surface facilities.

CONCLUSION

The Hume Coal Project is on track for public exhibition of its EIS early in 2017. Mine plan design has been influenced by the geological model and the interpreted geological structures as well as other constraints. Coal resources are sufficient to justify an underground operation centred around development and maintaining ground stability through pillar design. This stability will allow protection of the overlying groundwater system and critical surface infrastructure.

A significant portion of the Narrabeen and the Illawarra Coal Measures are not present within the Authorisation 349 area. It is uncertain, if the Narrabeen strata was deposited in the first instance or if the material was eroded by the Hawkesbury Sandstone.

The Wongawilli Seam is of sufficient quality to produce a good coking coal as well as a middlings thermal product. The mine's position is a significant advantage in respect to infrastructure with both major road access and the use of existing rail for coal transport to Port Kembla's coal loader.

Adverse community groups are opposed to the project, but the commitment that Hume Coal has to the consultation process is seeing changing attitudes in the region. A strong community consultation program by the company has seen local support for the development and jobs improve significantly.

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A CASE STUDY ILLUSTRATION OF THE LAYOUT AND SUPPORT IN AN INDIAN COAL MINE WITH DIFFICULT ROOF CONDITION WHERE CONTINUOUS MINER IS IN OPERATION

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ABSTRACT: Coal is the most important source of energy. Coal India Limited (CIL), a public sector coal company of the Ministry of coal, has reported the highest ever production of 538.75 MT of coal during 2015-16. This performance was made possible through better planning and monitoring of mining projects, increase in productivity, and better utilization of machinery and commissioning of new projects. In near future CIL will be concentrating on underground mining. In 2013-2014, 34.357 MT of coal was produced by CIL from underground mines. Continuous miners are being used for enhancing production in Tandsi underground coal mine of Kanhan Area of Western Coalfields Limited in difficult roof condition. In this paper the layout of the mine was studied and the support system was also considered. To meet the demand from underground coal production, more mines in India should install continuous miner systems and thereby improve both production and productivity. But detailed analysis of the layout and support system should be done beforehand as this has not been as successful as expected especially in Tandsi seams. There were numerous bottlenecks in the system.

INTRODUCTION

Coal is the most important source of energy for electricity generation in India. Coal India Limited (CIL), a public sector coal company of the Ministry of coal, has reported the highest ever production of 538.75 MT of coal during 2015-16 which is 44.52 MT more compared to 2014-15. So it is showing a production growth of 9%. That was the highest ever increase in coal production. Production of 2014-15 was 494.23 MT (Ministry of Coal 2016). This performance was made possible through better planning and monitoring of mining projects, increase in productivity, and better utilization of machinery and commissioning of new projects. Productivity is measured in terms of raw coal output in tonnes per man-shift (OMS). There has been substantial improvement in OMS in CIL group of Coal Companies during the last decade (Hekimoglu and Ozdemir 2004). As against an OMS of 0.58 tonnes at the time of nationalization, OMS in Coal India Limited during 2014-2015 was 6.05 tonnes and 6.65 tonnes in 2015-16 (provisional). In SCCL the OMS during 2002-2003 was 1.89 tonnes and during 2014-2015 was 4.20 tonnes. This change in productivity was possible by mechanization. Now CIL will be concentrating on underground mining. In 2013-14, 34.357 MT of coal was produced by CIL from underground mines. Technology wise coal production (in Mt) of CIL in 2013-14 was as follows:

- 1. Bord and Pillar Method
 - 1. a. Drill-Blast
 - Intermediate technologies with side discharge loaders 14.74
 - Intermediate technologies with load haul dump machine 16.77
- 2. Continuous miner technology 2.42
- 3. Longwall / Shortwall technology 0.057
- 4. High wall technology 0.37

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Continuous miners are being used for enhancing production from underground coal mines. At present CIL uses continuous miners in eight underground projects with a total capacity of 3.35 MT a year. CIL is introducing continuous miners with a total capacity of 13.5 MT in 19 more mines. In addition, introduction of continuous miners in nine more mines with a total capacity of 10 MT is under consideration (Bose 2015). Continuous miner playing a major role in the underground coal mining industry due to their safety as well as high productivity. Regarding past experiences in Indian coalfields, Chirimiri and Jhanjra mines have achieved exceptional performances, with a production of 5,00,000 tonnes per year. Tandsi mine has difficult roof conditions and does not typically achieve high production and productivity. The difficult roof conditions have been controlled with proper roof bolting systems and an output of 1200 tonnes per day has been achieved. There are many mines in India that may have potential and economic viability with proper ground control technology concepts for installing continuous miner systems. In Tandsi Mine of Kanhan Area of Western Coalfields Limited continuous miner was installed to meet the demand for coal.

OBJECTIVE

- 1. To study the layout in a mine where continuous miners have been installed in India with difficult roof condition.
- 2. To study the support system in an underground coal mine where continuous miner is in operation with difficult roof conditions.

GENERAL INFORMATION ABOUT THE CONTINUOUS MINER

Room and Pillar mining technique with continuous miners is the pre-dominant technology used in underground coal production. Room and Pillar mining method falls in the category of open stopping method and is often used to excavate horizontal deposits with reasonably competent roof strata. A typical continuous miner is shown in Figure 1. A continuous miner section is equipped with a continuous miner, 2-3 shuttle cars, a roof bolter and a scoop for auxiliary jobs. The continuous miner extracts the coal from a heading, while the roof bolter installs roof bolts in the immediate excavated heading. Shuttle cars work in tandem with the continuous miner and load the coal on to the feeder breaker arrangement which in turn loads the material on to a conveyor which is usually a high capacity conveyor (Burgess-Limerick and Steiner 2013). As shuttle cars are operated electrically the cables restrict the flexibility and hence the optimal routes should be considered. The out bye transport system should be made of suitable capacity so that the output is optimised.



Figure: 1 Continuous miner

Typical specifications of Joy 12CM continuous miner are:

- Length of the continuous miner is about 16.5 m
- Capacity of cutting is around 1000 t / hr.
- Weight of the continuous miner is around 75 t.
- Mining height ranges from 1.56 -3.76 m.
- Cutting power of the continuous miner is around 2 x 245 kW.

- Total power of the continuous miner being around 750 kW, including pump, gathering head and Traction.
- The ground pressure of the continuous miner is around 234 kPa.

The supporting is usually done with 'spin to stall resin bolting' using quad bolter or the so called multibolter as shown in Figure 2, in tandem in the entry which has been worked out (Singh 1998). The multi-bolter/quad-bolter has a twin boom with four bolting rigs.



Figure: 2 A typical Joy Quad bolter (Multi-bolter)

Typical specifications of the multi-bolter is:

- Height of the multi-bolter ranges from 2.4 m 4.7 m.
- Drill rigs feed around 25 m / min for drilling holes for roof bolting.
- Drill motor torque is around 384 Nm (max).
- Weight of the multi-bolter is around 30 t.
- Power of the multi-bolter is around 112 kW.

The continuous miner extracts coal in around two pass operation which means it changes its position to cut the full width of the entry (around 5-6 m). The length of the cut (6 to 12 m) varies depending on the roof conditions as well as the ventilation requirement and the features of the continuous miner.



Figure 3: Five heading mechanized continuous miner layout

Five heading mechanized continuous miner layout is shown in Figure 3. In this technology, mining takes place by place changing system. Five heading panel is optimum. The Continuous miner cuts

and load coal to shuttle car at a place. For developing full width of Gallery, continuous miner cuts in two passes at a place. The shuttle car hauls the load to the feeder breaker. Feeder breaker feeds sized coal to the gate-belt conveyor at a consistent rate. After completing a cut of desired length (cutout length) continuous miner moves out of the place and the roof bolter moves in for roof bolting. Same sequence of operation is repeated at another place. A typical cutting sequence for development Panel and depillaring panel is shown in Figure 4 and Figure 5 respectively.



Figure 4: A typical cutting sequence for development panel



Figure 5: A typical cutting sequence for depillaring panel

ADVANTAGES AND DISADVANTAGES OF CONTINUOUS MINER

The basic advantages of working with continuous miner are as follows:

- The production in this case is higher which should be matched by suitable out-bye transport system.
- Mechanised system, so no problem or delay in work.
- OMS higher as production and productivity greater.
- No drilling blasting required as cutting of coal done using the continuous miner.
- Loading usually done by shuttle car followed by haulage transport system thus it is much faster.

The basic disadvantages of continuous miner are as follows:

- The continuous miner cannot work in a watery seam.
- Strata condition is to be good as it requires a good gallery width.

- In place of normal rectangular pillar, round corner pillars are required.
- It is also not suitable for undulating floor.
- Good out-bye matching transport is required with high capacity shaft/skip arrangement to take coal to the surface.

Thus continuous miner installed in number of collieries has worked with relative success at some of the mines in Western Coalfields Limited and South Eastern Coalfields Limited as well in S.C.C.L.

CASE STUDY REGIONS

The case study region selected is Tandsi Mine of the Kanhan Area in Western Coalfields Limited – a subsidiary of Coal India Limited.

DETAIL LAYOUT AND METHOD OF WORKING

The mine there is being worked by Bord and Pillar method in which continuous miners are being used as the main equipment for extracting coal. The Pillar size which is being kept for support of the overburden pressure is $35 \text{ m} \times 35 \text{ m}$, $40 \text{ m} \times 40 \text{ m}$ and $40 \text{ m} \times 35 \text{ m}$. The depth of cover above the seam varies between 220 m to 400 m. The gallery width being used for continuous miner production is $3.6 \times 2.7 \text{ m}$; $4.8 \text{ m} \times 2.7 \text{ m}$ and $4.8 \text{ m} \times 3.5 \text{ m}$. This roof of the mine is usually of sandstone but the presence of cross-bedded layers and joints, which cause immediate roof collapse, resulting breakage which varies from 4 m to even up to 8 m deep into the roof.

The coal present in the mine is soft Bituminous coal. The problems faced by the mine is usually heavy strata control problems which has resulted in these roof falls and thereby reduce the average production per day to less than 500 t. The mining potential of this mine is seriously affected.

The production seen with the continuous miner did not reach 500 tonnes/day average production and hence the mine incurred heavy losses. The production with continuous miner in the year 2002-2003 was around 50900 t and in 2003-04 was around 192 400 t. Figures 6 - 8 show various layouts.



Figure 6: Layout No.1



Figure 8: Layout No.3

SUPPORT SYSTEM

The main cause of low production was geologically disturbed strata conditions. Roof cracks were observed at the early stages of development and hence this resulted in roof falls at many places. The roof in the case of Tandsi Mine was having a Rock Mas Rating (RMR) of 44-64. The roof was supported by using roof bolts up to 1.5 m in length and using spin to stall resin bolting using cement capsules.

As shown in Figure 6 (Layout no.1), the system initially designed to support the roof for a gallery width of 4.5 m and the height of the gallery being 3.0 m. The roof bolting system (1) in this case is shown in Figure 9. The roof bolts were installed such that there were three bolts in each row. These had a space of 1.5 m among each other and 0.75 m from the pillar. One of the bolts was drilled vertically. And the other two were so drilled that they were inclined in such a way that they went into the pillar there by supporting the overburden pressure. This resulted in heavy roof fall, resulted in changing the support design which can hold the roof better.

Due to failure of the above pattern a new layout as shown in Figure 7 (Layout no: 2) was developed in which the gallery width was reduced to 3.6 m and the height was kept constant at 3.0 m. The roof bolting system (2) was also changed as shown in Figure 10. In this case also we had three bolts which were installed in a row. One of the bolts was drilled in vertically upwards and the other two were drilled in, an inclined direction into the side of the pillar. The spacing between the bolts was kept at 1.5 m and the distance between the side pillar and the bolts was 0.3 m. But this modified design also resulted in roof falls.

At the colliery level the support design was further modified as shown in Figure 11. In this design the number of holes in each row was further increased. Corresponding layout design was shown in Figure 8 (Layout no.3). The gallery width was kept constant at 3.6 m width and 3.0 m height. The supports increased to 4 roof bolts. The distance between the holes being kept at a distance of 1.0 m each and the distance between the bolts and the pillar being kept at 0.3 m distance. The 2 bolts being drilled in were drilled in vertically and the other two were kept inclined at an angle and drilled at the side into the pillar. Roof falls being prevented to an extent and this layout was finally adopted.



Figure 9: Support system design (1)



Figure 10: Support system design (2)



Figure 11: Support system design (3)

CONCLUSION

To meet the demand from underground coal production, more mines in India should install continuous miner systems and thereby improve both production and productivity. Some underground mines of Coal India Limited have achieved exceptional performances whereas some mines of Western Coalfields Limited have difficult roof condition.

The difficult roof condition have been controlled with proper roof bolting technology and output up to 1200 tonnes per day have been achieved. But detailed analysis of the layout and support system should be done beforehand as this has not been successful as expected especially in Tandsi seams. There were numerous bottlenecks in the system, which have to be addressed. Roof condition was one of the major problems which have been partially addressed. The correct implementation of these systems would make a significant impact upon safety as well as providing a major increase in production and productivity in underground coal mines.

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FINITE DIFFERENCE MODELLING IN UNDERGROUND COAL MINE ROADWAYS

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ABSTRACT: This paper presents stability analysis of roadways of the Tabas coal mine in Iran. Tabas Coal Mine is the first fully mechanised coal mine in Iran, producing 1.5 million tons of coal per year. The mine extracts coal by both longwall and room and pillar methods. The results gathered from field investigations and the geomechanical properties of rocks, were determined in the laboratory and indicate that the rock masses of this area are weak. So, the excavated roadways need to have suitable support. For this purpose, the roadways were modeled with FLAC-2D software. The Finite Difference Method (FDM) models were calibrated to study the interaction between rock mass and support. The use of V29 and V36 section arches are under consideration. After modelling these roadways in FLAC^{2D} software the results achieved from this model show that; displacements of around the roadways are high and safety factors are very low, so roadways need to be support. The extracted results from this software show that; steel arch V36 with a spacing of 1m is the best support system for these roadways. With this type of support system, displacements around the roadway are low and safety factors are in suitable values. After installation it was observed that the critical strain values on roadway walls and roof were less than the permitted values, which demonstrated the roadway stability.

INTRODUCTION

Coal is one of the most important minerals used in many smelting factories of Iran. Most of the coal mines of Iran are located in the Tabas Coal field located in the east of Iran. Most of these mines have difficult geological conditions. Most of them have weak rocks with low thickness and high slopes. With these conditions excavated roadways in these coal seams usually have many support problems. Tabas coal mine is one of the mines that provide coal for Isfahan Iron smelting factory of Iran. For exploiting of this mine Longwall mining method is used. In this research the geomechanical conditions of these rocks and the stability of the main-roadway in the mine were studied. Roof stability issues and the development of cavities in the immediate roof are a key concern for underground longwall mines. New technologies have enabled a greater knowledge and understanding of geological factors and *in situ* stresses in an underground environment, leading to a more accurate prediction of roof stability. These facilitate a safe working environment, which is imperative to all mining operations.

LONGWALL MINING

Longwall mining is the most common method of underground coal extraction used in the world today. Longwall mining extracts coal in large rectangular blocks, defined during development, in a single continuous operation (Aziz, *et al*, 2007). Each block of coal, known as a panel, is developed by driving a set of headings on either side of the panel off the main access roads. The start of the working face is created by the joining of these roadways. The longwall face is supported by hydraulic roof supports, whose main function is to provide a safe working environment as the coal is extracted and the longwall equipment advances. A goaf is formed as the immediate roof is allowed to collapse behind the mined out area. Figure 1 shows a schematic of a typical longwall retreat method. The thickness of a coal seam is another major contributing factor in the selection of the longwall mine design. Economically, maximising the recovery in thick seams can prove to be highly beneficial; however

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mining thick coal seams can often lead to roof stability issues that must be alleviated to maximise the benefits.

SEAMS PROPERTIES

Tabas underground coal mine is located about 85 km on the southern area of Tabas, Birjand Province, Iran. This mine is the first fully mechanised coal mine in Iran and produces 4000 tons of coal per day. The East 2 longwall panel has a face width of 180 m and panel length of 1200 m. Figure 2 shows the location of the extraction panels (East1 and East2). The C1 working seam thickness varied from 1.8 to 2.2 m with dip varying between 11° and 26°. The roof of the coal seam contained 0.1- to 0.2 m mudstone, siltstone/sandstone interfaces, and sandstone. The C1 seam had a uniaxial compressive strength of less than 5 MPa. The other seams near the C1 seam were C2 and D1 above, and B1 and B2 below (IRITEC, 1992). The studied roadways were the main roadways of this mine. This roadway is located 350 m depth from surface with 2.2 m average thickness and 20° stone, siltstone, silty sandstone and sandy siltstone. Table 1 displays the intact rock and rock mass properties such as uniaxial compressive strength, the material constant and the geological strength index.



Figure 1: Longwall retreat mining (Aziz *et al*, 2007)

Figure 2: Extraction panels of East 1 and East 2 (IRITEC, 1992)

Depth into Roof (m)	Intact R						Rock Mass				
	UCS (MPa)	m*	GSI	m	S	C (MPa)	Ф (deg)	Tensile Strength(Mpa)	Compressive Strength(Mpa)	E (Gpa)	υ
Coal	5	1	25	0.1	0.0002	0.1	20	0	0.5	0.7	0.3
0 - 2.12	32	7	31	0.6	0.0004	0.5	21	0.02	0.5	1.7	0.3
2.12 - 3.35	73	13	39	1.5	0.001	1	44	0.05	2.2	3.5	0.3
3.35 - 3.8	32	7	36	0.7	0.0008	0.6	31	0.03	0.8	2.7	0.3
3.8 - 4.75	73	13	39	1.5	0.001	1	44	0.05	2.2	3.5	0.3

Table 1: Intact rock and rock mass parameters (IRITEC, 1992)

STABILISATION

The usual support system for coal mines in Iran is steel arches, so a suitable arrangement for this kind of support system must be designed. The pressure that steel arches induct to walls and roof of roadways must be calculated. For this purpose Table 2 is used (Hoek, 1999). In coal mines of Iran, usually steel arches of type TH section is used (Table 2). To finding the best support system from Table 2, different types of TH section steel arches (different profile number) with different spacing are used. After finding the support pressure, it is applied to FLAC-2D software. The extracted results from this software show that; steel arch V29 and V36 with spacing of 1 m is the best support system for these roadways.

Support type	Flange width - mm	Section depth - mm	Weight - kg/m	Curve number	Maximum support pressure <i>p_max</i> (MPa) and average maximum strain <i>s_{max ov}</i> for a tunnel of diameter <i>D</i> (m) and a support spacing of <i>s</i> (m)
	305	305 203	97 67	1 2	$p_{i\max} = 19.9D^{-1.23}/s$ $p_{i\max} = 13.2D^{-1.3}/s$
Wide flange rib	150	150	32	3	$p_{i_{\text{max}}} = 7.0 D^{-1.4} / s$ $s_{\text{max}, av} = 0.30\%$
I section rib	203 152	254 203	82 52	4	$p_{i \max} = 17.6 D^{-1.29}/s$ $p_{i \max} = 11.1 D^{-1.33}/s$ $s_{\max, ov} = 0.25\%$
The section of	171 124	138 108	38 21	6 7	$P_{i \max} = 15.5D^{-1.24}/s$ $P_{i \max} = 8.8D^{-1.27}/s$ $s_{\max, ev} = 0.55\%$

Gate roadways were driven with a 12 m² cross section, using yielding steel sets of 29 TH profile with a spacing of 1.2 m (Figure 3). Figure 4 shows a gate roadway at A station without deformation. The support system was yielding steel sets with a lower segment (Figure 3) and spacing of 1.00 m.



Figure 3: TH V29 steel set support (IRITEC, 1992)



Figure 4: The roadway East 2 roadway profile (IRITEC, 1992)

SIMULATION OF YIELDING STEEL SUPPORT

Due to the complexity of the behaviour of the yielding steel set support, some simplifications are always necessary to simulate it by a pressure inside the excavation. It is based on very simple assumptions (Figure 5). Analysing the top of the support, it can be seen that an axial load at the legs clamps F (where the segments slide) produces a constant pressure p against the rock mass over the crown of the arch given by:

$$p = \frac{F}{a \times s} \tag{1}$$

where a is the radius of the excavation and s the steel sets spacing. If F_{lim} is the tangential load which causes sliding of steel segments, then the internal pressure equivalent to the steel set action when it is yielding is:

$$p_{lim} = \frac{F_{lim}}{a \times s} \tag{2}$$

In the horizontal direction, i.e. analysing the right part of the support, the value of the constant pressure due to a load F, is:

$$p = \frac{F}{2 \times a \times s} \tag{3}$$

then the equivalent pressure is $p_{lim}/2$.

The value of F_{lim} for the used steel set type estimated from laboratory test is 250 kN, but a more realistic value of 230 kN has been used. Taking into account that the half width of the roadway is 2.25 m, the radial pressure equivalent to the support when the steel sets spacing is 1.20 m is 0.085 MPa. If the steel sets spacing is 1.00 m, then the pressure is 0.10 MPa.



Figure 5: Basic assumptions to estimate the pressure equivalent to a yielding steel set (Torano *et al.* 2002).

TH arches are of the yielding type and the load capacities quoted from the test results are obtained by ensuring that the yield clamps do not slip. This is usually achieved by welding them together. In underground use, of course, the clamps can slip and this type of arch is designed to close (i.e. reduce its internal cross section) as load is applied. Yielding arches can accept a higher degree of strata movement than conventional rigid arches. However, high lateral movement or eccentric loads can result in the clamps locking which can lead to early failure of support. An even load distribution around the arch is critical if optimum performance is to be achieved with a TH arch. It could be argued that a long life decline is the very place where yield and hence closure cannot be tolerated. In this case, a strong, rigid arch would be preferable (MRDE, 1970).

NUMERICAL MODELLING

Computer modelling in the rock mechanics field has undergone significant developments in recent years. Developments in software and computers, allied with more sophisticated measurement of rock parameters, now allow the rock mechanics engineer accurately to simulate ground conditions and behavior (Garratt, 1997). Consequently computer numerical modelling now predominates as the method used by Rock Mechanics engineers to design support and reinforcement patterns for underground openings, including mine roadway support. The Finite Element method and Finite Difference method are alternative analysis techniques both based on discretising the domain to be analysed.

FLAC-2D stresses and displacements induced by underground excavations. FLAC solves a wide range of mining and civil engineering problems. Materials in the model can be linear elastic and nonlinear (Mohr–Coulomb and Hoek–Brown failure criterion), and discontinuities may be incorporated into the model. This feature was used to model the movements of blocks in the roadway (Itasca, 2004). FLAC is a two-dimensional code and these problems were analyzed on the assumption of plane strain along the axis normal to the plane of the model. This is of course an approximation and prevents the investigation of some important factors. In the case of roadways for example the assumption of plane strain implies that one of the principal stresses relative to a mine roadway can have an important influence on its behavior (Itasca, 2004). The first step of numerical analysis with FLAC-2D software is modelling of underground openings in a computer. In this part the model boundaries, *in situ* stresses, boundary conditions, material properties, and creating the finite element meshes are discussed.

Geometry of the model

The geometry of the area modelled was 40 m by 40 m with a roadway width of 4.5 m and height of 3.5 m. The coal seam was modelled as 2 m thick and dipping at 20°. The roadway immediate roof stratification sequence consisted of siltstone and sandstone above the roof. The geometry of the model defined is shown in Figure 6.

Boundary conditions

The model assumes plane strain state, nil displacements at the boundaries and constant field stresses. If the model is used to simulate convergence without longwall influence, then it is assumed that only the stress due to the pressure of the overburden at that depth is acting.

In situ stresses

Measured values for the *in situ* stresses were available from existing measurements at the mine, the closest being at the outbye end of the preceding district roadway. The measured stress direction is consistent and places the gate roads in a favorable direction approximately in line with the maximum horizontal stress. There is more discrepancy between the measured horizontal stresses magnitudes. Giving greater significance to the nearest measurements, the lateral stress acting across the gate roadways is expected to lie in the range 5-10 MPa. The depth of cover increases to approximately 350 m at the inbye end of the district. A representative value of 13 MPa was therefore chosen for the vertical stress component.

The critical strain

Here, the Sakurai method (Sakurai, 1997) was used to investigate the roadway stability. The method evaluates the critical strain in the elastic region. Since the rock mass is under triaxial stress, using the maximum critical strain for investigation of roadway stability is sensible (Sakurai, 1970). They suggested Eqs. (4) and (5):

$\log \epsilon_c = -0.25 \log E - 1.22$	(4)

$$\gamma_{c} = (1 + v)\varepsilon_{c} \tag{5}$$

Where; E= Young's modulus of intact rock($\frac{\text{kgf}}{\text{cm}^2}$), ϵ_c = critical strain in uniaxial strenght compressive, γ_c = critical strain, V = Poisson's ratio.

Critical displacement values based on the critical strain are obtained by following equation (Sakurai, 1997:

$$\varepsilon_c = \frac{U_c}{a} \tag{6}$$

Where; U_c = Allowable displacement; a = radius of the roadway. In No support state, it can be also seen in Table 3, which shows the maximum displacement values and in Table 4. The critical strain values over the walls are more than the allowable strain values, which indicate roadway instability. Table 5 shows properties of V29 and V36 steel arch.

Model	Roof	Floor	Right hand Rib	Left hand rib	
No Support	18.9	17.3 51.3		66	
TH Arch V29	4.5	5.98	3.3	8.96	
TH Arch V36	3.6	4.23	2.8	6.65	
Critical Displacement	10.2	10.2	18.3	18.3	

Table 3: Horizontal and vertical displacement of around East 2 roadway (mm)

Table 4: Maximum shear strain increment around East 2 roadway (*10^-3)

Model	Roof	Floor	Right hand Rib	Left hand rib	
No Support	4.35	40.6	55.3	94.4	
TH Arch V29	1.26	2.16	2.93	1.31	
TH Arch V36	0.95	2.02	1.75	1.10	
Sakurai Shear Strain	4.54	4.54	6.48	6.48	

Result of numerical modelling with FLAC 2D software that using TH Arch V29 and TH Arch V36 for support of East 2 roadway is very good. As observed the critical strain values on roadway walls and roof are less than the permitted value which demonstrated the roadway stability. The geometry of the model defined is shown in Figure 6. In Figure7 shows coal layer deformation due to the increasing stress around East 2 roadway. The total displacements of ribs of roadway are shown in Figure.8. Figure 9 shows vertical displacement after steel arch V36 support in East 2 roadway that has good agreement with experimental results.

Arch Type	Wx (cm³)	lx (cm⁴)	Area (cm³)	Width (mm)	Height (mm)	Weight (kg/m)
TH Arch V29	94	616	37	151	124	29
TH Arch V36	102	618	41	162	138	36

Table 5: V29 and V36 steel arch properties (IRITEC, 1992)





Figure 6: Model roadway profile, layers, and V36 arcs numerical modelling



Figure 7: Total displacement around the roadway (mm)



Figure 8: Displacements horizontal of right and left hand rib (mm)



Figure 9: Displacement vertical at different levels (mm)

CONCLUSION

This research is about stability analysis of East 2 roadway in Tabas coal mine of Iran. The results extracted from software show that; displacements around the roadway are high and safety factors are low, so the roadway needs to have a support system. To find the best support system, different types of TH section steel arches (Different profile number) were modeled. The extracted results show that steel arch V36 with spacing of 1 m is the best support system for this roadway. With this type of support system, displacements around the roadway are low and safety factors are of suitable values. From the numerical simulations the following conclusions can be inferred.

- Measurement devices for roof movement must be capable of detecting roof movement over the required range.
- Enough measurement devices should be installed to minimise the risk of undetected movement occurring between measurement points.
- A measurable level of roof movement must occur before ground control failure.

• Roof movement must be measured sufficiently in advance of ground control failure to allow effective early warning.

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SYSTEM FOR CREATION AND DISPLAY OF 3D MAPS OF COAL MINES

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ABSTRACT: This paper presents a system for creating, processing and visualisation of 3D maps of underground coal mines (old or new mine shafts with no other types of maps available, corridors affected by an accident). The maps in the form of point clouds are created by 3D laser scanning. The paper mentions the most important algorithms applied in point cloud data pre-processing, especially voxelization, outlier removing and smoothing. In more detail is described the rendering engine with its advanced visualization methods like shading and colouring and special distance measuring and shape highlighting features designed to help the user to orient in the virtual view of the mine.

INTRODUCTION

Accidents in underground coal mines are often very grave and lethal. Especially serious are underground explosions caused by dangerous concentrations of coal dust or methane. Typically, the area of the mine affected by an explosion is sealed by a thick stopping with a service hole and human rescuers are not allowed to enter the zone until the monitored parameters drop below critical limits. It would be extremely beneficial, if the rescuing operations could start as soon as possible, because every wasted human life is a huge tragedy.

A logical solution is to use mobile robots, which could be applied much earlier. Mobile robots can perform reconnaissance of the tunnels to measure concentrations of dangerous gasses and temperatures and provide information about the physical state of the tunnels. The research in this area has been already running for many years (for example Ray, *et al*, 2015; Gomathi, *et al*, 2015).

A new research project "System for virtual TELEportation of RESCUER for inspecting coal mine areas affected by catastrophic events (TeleRescuer)" has started in 2014 in the framework of an EU programme Coal and Steel under the grant agreement RFCR-CT-2014-00002. This project is solved by multiple teams from the Czech Republic, Poland and Spain and the goal of the project is to develop a mobile robot possessing all useful tools and features for this task in one body and – which is unique – to get the important certifications for deployment in coal mines without risks of causing additional damage (eg, explosion safety), as described by Novak, *et al* (2015a); Novak, *et al* (2015b); Moczulski, *et al* (2014).

PREPARATION OF A POINT CLOUD BY SCANNING

The mobile robot TeleRescuer is equipped with a huge variety of sensors, including a 3D scanning device consisting of the Sick LMS111 2D laser range finder mounted on a turning platform providing the third dimension (Figure 1). To make one 3D scan, the scanning device performs a series of 2D scans with increments of the turning angle. The result is presented as a set of points in spherical coordinates. These individual 3D scans can be made in multiple places and then can be merged into one complex map of the mine in the form of a point cloud (for example Blanco, 2013; Rusu, *et al*, 2011; Universität Karlsruhe, 2011).

Distance between the scans is typically a few meters (Olivka, *et al*, 2016). The density affects precision of the resulting map and the amount of details, but also the data size, which can be a limiting factor when the 3D map has to be transferred wirelessly to an operator in real time.

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Figure 1: Model of the ED scanning device

INPUT DATA FILTERING

For efficient processing and display of the point cloud without excessive errors it is very important to perform some filtering of the input point cloud. All the following algorithms were implemented using the *Point Cloud Library (PCL)*, which is a very powerful library for point cloud and 3D geometry processing (PCL, 2016b).

Decreasing number of points

The Laser Scanner Sick (LMS) 111 has a working range of 20 m. Because the 3D scanning works with spherical coordinates, density of the measured points lowers with distance from the origin and individual 3D scans must be done much more often than just every 20 m; a good value discovered by testing is between 2 and 5 m. This means that the scans overlap, which is beneficial for increasing details of the resulting point cloud, it also dramatically increases its size and complexity. Multiple points can even occupy almost exactly the same 3D coordinates, which is a waste of data and processing power.

A common way of removing very close points is *voxelization*. The corresponding implementation in the :Point Cloud Library (PCL) is a filter called *Voxel Grid* that works by presenting a 3D box (voxel) grid over the points; then in each box all points are approximated by just one, with location within the box calculated as average of the removed points (PCL, 2016a). This filter in general is used for down sampling, i.e. lowering of density of a point cloud while preserving the overall distribution of points in space. With a small voxel size (approx. 10 mm in our case of mine tunnels), no details are lost and only duplicate points are removed. In testing with scans crated every 2 m and 10 mm voxel size, the point cloud was reduced by 9 % of points.

Removing noise

One of the typical problems of point clouds acquired by 3D scanning is noise caused by measurement errors. Measurement errors can corrupt the point cloud with sparse points lying out of larger clusters of points and not representing real objects. This complicates further processing, especially analyses like estimation of surface normal, and can lead to false values.

PCL contains the filter called *Statistical Outlier Removal* that works by performing statistical analysis on each point's neighbourhood and trimming those which do not meet certain criteria (PCL, 2016c). The optimal configuration of this filter in testing removed around 0.8 % of points.

Another negative effect of noise in the point cloud data caused by measurement errors is when what should be a smooth surface is actually represented as rough. This can be dealt with by smoothing, for example using the *PCL* algorithm *Moving Least Squares* surface reconstruction method (PCL, 2016d). This resampling algorithm attempts to recreate surfaces by higher order polynomial interpolations between the points. A side effect of this algorithm is the ability to calculate also surface normal vectors, which will later be used for lighting. The important configuration value is the search radius. Optimal value is 100 mm, demonstration of the impact of the value is shown on Figure 2 – too high a value can erase important details.



Figure 2: Comparison of two different smoothing settings – radius 100 mm and 300 mm

Graphical engine

The goal of the TeleRescuer project is to provide "virtual teleportation" of a human rescuer into the sealed mine. Thus, the point cloud should be presented to the operator in a clear and illustrative way and allow him to inspect all important parameters in real time while roaming freely through the shafts in a virtual space, or while watching the actual surroundings around the mobile robot for safe driving with remote control. It also should integrate important sensor readings and similar data into the point cloud.

The best solution was to create a custom graphical engine tailored to the needs of this project. The engine is implemented in C++ and uses the Direct3D 9.0c graphical API for hardware graphic acceleration of many complex calculations related to rendering.

Rendering of individual points

Every point in the point cloud stored in a special PCL array contains the following parameters:

- Position vector $\mathbf{p} = (p_x, p_y, p_z)$.
- Normal vector $\mathbf{n} = (n_x, n_y, n_z)$.
- Colour vector $\mathbf{c} = (r, g, b, a)$.

The points with these data are expressed as *vertices* and stored in a Direct3D *vertex buffer*, which is then drawn as a Direct3D primitive type *point list*. During the rendering process of Direct3D, every vertex is processed by a *vertex shader* (a programmable stage that can modify parameters of each vertex in many ways, but at least have to transform 3D coordinates to 2D screen coordinates) and by a *pixel shader* (this stage calculates the colour of the resulting pixel).

Performance optimisations

Even hardware-accelerated rendering of the very simple *point list* primitives would not be able to display hundreds of millions of points with an acceptable framerate on a mid-level computer. It is necessary to lower the number of actually processed and drawn points in a way that will not negatively affect the displayed information. The two implemented methods are:

- View frustum culling ignoring points out of the current camera view,
- Level Of Detail (LOD) drawing fewer points in larger distances from the camera.

These algorithms cannot be efficiently calculated for every individual point. The applied solution utilizes the Octree system (Wikipedia, 2016) with one hierarchy level to partition the point cloud into a set of box nodes – Figure 3. View frustum culling and LOD are then applied on the level of nodes rather than points.



Figure 3: Simplified Octree system with one hierarchy level

Tests demonstrated that view frustum culling removes 70% of points in average, because typical view is from inside the mine shafts looking towards one direction and most points are behind the view. That however still leaves very large number of points, especially when the operator is looking down a very long corridor (in some extreme situations view frustum culling may remove even less than 10 % of points). The LOD algorithm calculates distance of each node from the virtual camera and this value is used to determine how many percent of its points will be drawn. When properly configured, this optimization is not visually detectable at all because far points blend together anyway.

With both the optimizations active, the actual number of rendered points typically never rises above 10 % of existing points, but usually the ratio is even much lower (around 2%).

Projection transformation

For good visual impression it is necessary to draw the point cloud with perspective projection, which is easily achieved in Direct3D by using a 4x4 homogenous perspective projection matrix in the vertex shader stage. By default, every individual point rendered as a Direct3D point lists occupies exactly one pixel on the screen, regardless of its distance from the camera (the projection matrix does not apply here). Distances between points are affected by perspective foreshortening but size of the points is not, which makes large gaps between points close to the camera and the operator loses the impression of being close to a wall – this can be very dangerous for direct control of the robot (Figure 4). The solution of this problem is to use the output parameter PSIZE of vertex shader, which controls the resulting pixel size of the point on screen. This way it is possible to get points drawn bigger than just 1x1 pixels with almost no additional cost. The value of PSIZE can be set individually for every vertex and is calculated in the vertex shader as the reciprocal value of the point's distance from the camera in 3D space.





Figure 4: Close view of a wall - rendered with fixed (left) and variable (right) point size

Point colours

The colour used to draw each point on the screen can be used for example to encode additional information from sensors (will be discussed in the following chapters) and another possibility is to use colours for better visual impression, which can help the operator to orientate in the point cloud representation of the real coal mine.

Two main characteristics of a colour – hue and brightness can be used. The resulting pixel colour of each point (Figure 5) in this software is calculated as:

$$\mathbf{c} = (0.7 \cdot \mathbf{c}_h + 0.3 \cdot \mathbf{c}_n) \cdot (a + d + s),$$

(1)

where:

- c_h colour vector (r_h , g_h , b_h) based on the absolute height of the corresponding point in the 3D space (its p_z coordinate). The colours generated by this algorithm create a linear gradient with key colours (from lowest p_z to highest): green yellow purple blue azure. The colours were chosen based on natural habit that ground is green (grass) and sky is blue.
- c_n colour vector (r_n , g_n , b_n) based on the normal vector of the corresponding point. This colour is a gradient between red, green and blue key colours (red when the normal is aligned with the *x* axis of the global coordinate system, green for the *y* axis and blue for the *z* axis).
- *a* ambient lighting scalar factor. Ambient light is constant in the whole scene and brightens up all points for better visibility.
- *d* diffuse lighting scalar factor. Diffuse light depends on the angle between the normal vector of the point and the vector of light rays hitting the point.
- *s* specular lighting scalar factor. Specular light depends also on the vector from the point to the virtual camera used to watch the 3D scene from and creates bright "reflections" of the light source.

The *d* and *s* factors work with geometrical properties of a virtual light source, which is a very simple directional light characterized only by a single vector or light rays (simulation of a light source located infinitely far away).



Figure 5: Rendering with point colours calculated by equation (1)

INTEGRATION OF ADDITIONAL INFORMATION TO THE POINT CLOUD

It is useful for the user to be able to get some additional information from the 3D map visualization, especially distance measurements or sensor readings.

Values acquired by sensors on the robot during scanning (such as temperature, wind speed, gas concentration) can be simply encoded by colour gradients and applied to points in the point cloud (Figure 6). When this functionality is activated by the user, the equation (1) is replaced with:

where c_s is colour vector (r_s , g_s , b_s) representing the particular sensor value (using a colour gradient).

$$\mathbf{c} = \mathbf{c}_{\mathrm{s}} \cdot (a + d + s),$$

Figure 6: Visualisation of temperatures in a tunnel using a colour gradient

Distance measurements are important to get quick details about for example the height or width of a narrow passage. The simplest distance measuring provided by the application works simply by clicking on any two points directly in the 3D scene – the points are then connected by a line, with distance measurement in meters (or millimeters) printed on the line. To get a better visual clue about the shape of a tunnel, it is possible to display a cross-section in red colour (Figure 7). This can help to discover tunnel branching, a hole or recess in the wall, or an obstacle in the tunnel.

(2)



Figure 7: Tunnel cross-section

CONCLUSION

The project is still in progress and the visualization software described in this paper is still in development, but all the mentioned functionality is already implemented and working. The images were prepared with a testing point cloud made from 11 individual 3D scans created in 2 m distances in a real coal mine in Gliwice, Poland. Total number of points after filtering is 2 112 270 and the number of nodes (boxes $2 \times 2 \times 2 m$) is 27 (Figure 7). Rendering runs at more than 250 FPS in 1920x1080 on Intel Core i5-3330 CPU and Nvidia GeForce 750 GTX. This leaves a lot of power reserve for even many-times larger point clouds.

The system is applicable as a general point cloud visualization and renderer, although it was designed primarily for coal mines with their special properties. Unique features of this system are: free walking or orbit camera, advanced rendering engine with illustrative colouring and lighting, sensor data integration, distance measurements and visualization and interactive cross-sectioning.

Further work will involve better integration of additional information, including a visual clue of the robot size for quick decision about its ability to travel through a narrow passage. The rendering engine is also currently being transferred into the Direct3D 11 API. This improvement will allow for example the use of *geometry shaders* (first introduced in Direct3D 10) for additional effects and optimizations.

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EXPERIENCE OF MONITORING SHEAR MOVEMENTS IN THE OVERBURDEN STRATA AROUND LONGWALL PANELS

Luc Daigle¹ and Ken Mills

ABSTRACT: Surface subsidence monitoring shows horizontal movements occur around longwall panels for a considerable distance outside the footprint of a longwall panel; typically several hundred metres to several kilometres. Less is known about how these movements are distributed between the surface and the mining horizon. A range of systems have been developed to measure how horizontal movements are distributed within the overburden strata generally and sometimes around specific geological structures. This paper describes the experience of using a range of these systems at various sites and some of the insights that these measurements bring with particular focus on the use of deep inclinometers.

The capability to measure induced displacements has developed over time from surface observations to use of borehole systems such as multi-arm callipers, downhole camera imaging and specially installed inclinometers placed to depths up to 300 m. Some techniques such as open boreholes and the multi-arm, oriented calliper have mainly been used at shallow depths where breakout and squeezing ground do not compromise the measurements. Others such as the borehole camera provide context but are not so suitable for quantitative measurement. The inclinometer installed in a large diameter borehole backfilled with pea-gravel has been found to provide high resolution measurements up to a horizontal displacement on any one horizon of about 60-80 mm. Inclinometers have been used at multiple sites around Australia to measure shear displacements to depths of up to about 300m. Shaped array accelerometers are an alternative that provide temporal resolution of a few minutes and provide continuous monitoring over a limited interval but tend to be most useful for monitoring the onset of low magnitude shear displacements.

INTRODUCTION

Surface subsidence monitoring provided the first indications that horizontal movements occur around longwall panels in response to mining coal over considerable distances from active mining (Reid 1991). Mills (2014) characterises these horizontal movements as being caused by systematic ground movements in response to vertical subsidence, horizontal stress relief towards the disturbed ground above the extracted void and horizontal movements caused by the interaction between the dilation of subsiding strata and surface topography. Oblique movements that include vertical and horizontal components are also observed around longwall subsidence where geological features such as reactivated faults and joints are present. Other discordant features such as both sub-horizontal thick massive sedimentary channels filled with coarse sandstone or conglomerate and intrusive sills (Daigle 2007) form distinct boundaries which can focus shear displacement.

Underground surveying in development roadways adjacent to active longwall panels, micro-seismic monitoring and observations of changes in horizontal stress magnitude and orientation around active longwall panels also indicate that horizontal movements occur at seam level and within the overburden strata in response to mining. An understanding of the nature and distribution of these horizontal movements within the overburden strata is helpful for interpreting various phenomena associated with mining. But getting measurements of lateral movements within the overburden strata is something of a challenge.

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Observations and measurements from within boreholes located around longwall panels provide perhaps the best opportunity to quantitatively study lateral movements as they develop within the overburden strata. These lateral movements can occur as general body distortions within the rock mass or as shear displacements on discrete planes such as bedding plane horizons, mining induced fractures or pre-existing geological structures.

Mills *et al* (2015) identify that shear displacements on specific shear horizons and associated with general body block movement occur in the direction of the major horizontal stress (i.e. in response to stress relief) to distances of at least 400m ahead of mining. Furthermore specific shear horizons are able to be correlated between boreholes a kilometre apart indicating that lateral displacements due to stress relief occur throughout the overburden strata for considerable distances. Additional shear distortion is observed to occur within close proximity (30 m or so) of the mining void. These shear distortions are consistent with the shear distortions caused by vertical subsidence and the redistribution of overburden load around the goaf edge.

Boreholes located adjacent to extracted longwall panels provide opportunities to observe and measure lateral ground deformation as they develop. This paper describes the authors' experience of using a range of borehole methods to observe and quantify the nature of lateral movements.

DOWNHOLE SURVEYS IN SHALLOW HOLES

Large diameter open boreholes provide a simple method to observe large scale shear movements at shallow depth. This approach is particularly suited to sites where large deformations occur or are expected to occur near the surface. If shear displacements are so large that the hole shears off, a new hole can be drilled and deformations can continue to be monitored.

At one site where a low angle geological fault was expected to move and did move significantly as a result of mining subsidence, large diameter open holes (250 mm to 300 mm in diameter) proved very effective as a means to monitor the anticipated reverse fault movement. Shear displacements were observed on multiple horizontal bedding parallel shear surfaces. The offsets observed in the holes were regularly measured using a borehole camera and multi arm calliper. Figure 1 shows a photograph of a shear displacement observed at one horizon.

As shear movements occurred, the shear offsets were observed by the borehole camera on each of the several shear horizons using the construction shown in Figure 2. The direction of an offset was identified using a downhole compass. Multi-arm calliper surveys provided a secondary measurement of the magnitude of displacement. Figure 3 shows how the data was combined, compiled and reported with successive surveys showing the progressive change in deformation and magnitude of change on each individual shear horizon.

Once the hole was fully offset by the fault, the camera and calliper tool could no longer pass the shear horizon, a new hole was drilled so that monitoring could continue.

Figure 4 shows the deformations observed in the geological setting at the site. The observed ground deformations indicate displacement was distributed over several bedding parallel horizons as the reverse fault was reactivated by mining subsidence. These studies demonstrated how large diameter holes deform and were effective in surviving large shear deformations. Understanding this behaviour enabled greater confidence with use and interpretation of inclinometers installed in large diameter holes to investigate anticipated shear deformations



Figure 1: Shear offset imaged in large diameter borehole.



Figure 2: Graphic method utilised to determine the offset magnitude and direction relative to the borehole.



Figure 3: Combined image survey data and multi-arm calliper surveys used to report the deformation of the observation borehole.



Vertical Displacement due to goafing

Figure 4: Shear displacement in observation boreholes.

INCLINOMETER MEASUREMENT SYSTEMS

Whilst the methods described above are suited to shallow open holes where large offsets are possible, an investigation method based on inclinometer monitoring is more suitable at greater depth where movements are smaller and may involve general body shear distortion that is not able to be observed using a borehole camera or calliper.

Inclinometer monitoring involves drilling a large diameter borehole in the area of interest through the full section where shear movements are expected to occur. The configuration is illustrated in Figure 5. A casing with two pairs of vertical grooves is installed in the borehole so an inclinometer probe can be lowered into the hole and withdrawn in steps of 0.5 m to get very accurate measurements of the incremental tilt in orthogonal directions. By repeating these surveys at regular intervals, incremental tilt is measured relative to the initial base survey and horizontal displacements can be integrated from these tilt measurements.

In some cases, the shear deformations become so large that the borehole shears sufficiently that the probe is no longer able to pass and further measurements are not possible. The survivability of the installation is dependent on the type of grout or backfilling used to hold the casing in place (Plinninger, Alber, Dullmann 2010). Vulnerability to shear displacements that preclude the probe passing is more common when shear occurs at specific shear horizons. Shear on specific horizons occurs more commonly within rock strata around coal mining activity whereas general body shear distortion occurs more commonly in soft soils and similar geotechnical applications.

Sensitivity of the inclinometer system to shear movement is improved if the annulus between the inclinometer casing and the borehole wall is backfilled with cement grout. However, this increased sensitivity comes at the cost of vulnerability to the holes being sheared off when shear displacements are concentrated at specific horizons. Grouting also requires control of the pressures differentials between inside and outside of the casing. Grouting of holes deeper than about 50 m typically requires staged grouting as the density differential between a cement grout outside the casing can become sufficient to collapse a water filled casing. However, stage grouting in deep holes can lead to sections of incomplete grouting where the inclinometer casing is not supported at all.

Backfilling with granular material has proved to be more effective method of backfilling in deep holes. Shear capacity of up to about 60mm of shear on a single shear plane are typically observed in a 200 mm diameter hole using this approach. The strategy is illustrated in Figure 6. Although there is some loss of sensitivity to first shear movements, this loss is more than compensated by increased shear capacity. A large diameter borehole (200-300 mm is common) is drilled and the annulus between the inclinometer casing and the borehole wall is backfilled with free-running pea gravel or similar material.

Use of adequately sized material allows the granular fill to sink effectively into position without bridging between the hole and inclinometer casing.



Figure 5: Schematic view of Inclinometer casing and Inclinometer probe.





Steel casing may be used to stabilise the large diameter borehole and prevent blockages associated with swelling clays or other types of borehole instability. The steel casing offers no resistance to ground movements but does help ensure that the backfill reaches the bottom of the hole and the inclinometer casing is thus fully supported.

The inclinometer probe measures the tilt on two orthogonal axes referred to as the A and B axes. The A axis is the primary axis of the instrument and readings on this axis are somewhat more accurate than readings on the B axis. The B axis measurement includes a component of tilt associated with the

wheels wandering in the grooves in the casing. This effect can be eliminated by making a second survey at right angles to the first.

Small, permanent offsets in the instrument alignment can become significant when integrated over the length of a deep inclinometer hole. These offsets can be eliminated by taking two sets of readings on each axis with the instrument reversed during the second set and averaging the results.

A set of four readings on each survey provides a single high resolution measurement with equal accuracy in each direction.

Some inclinometer casings are subject to small amounts of twist in the grooves that accumulate in deep holes so that the A+ axis varies down the hole relative to its orientation at the surface. A one off survey with a spiral probe is used in deep holes to determine how the orientation of the casing grooves varies down the hole. Spiralling of up to 120° have been observed in deep holes, so having a measure of the casing spiral is important in the context of resolving the direction of shear movements.

With accurate baseline measurements typically repeated before the onset of any ground movements to confirm the repeatability of the measurements and a spiral measurement, it is possible to resolve from the inclinometer monitoring the direction and magnitude of shear movements. In the context of ground movements around longwall panels, the direction of movement is significant in determining the cause of these movements. Movements that occur well in advance of longwall mining in the direction of the major principal stress are typically the result of *in situ* stress relief. Movements that occur in a downslope direction near the surface are more likely the result of topographic effects.

One of the limitations of inclinometer monitoring is that the surveys are labour intensive and can only be conducted infrequently. Focussing only on the section of borehole where movements are expected to occur or are of interest can increase the frequency of surveys that are able to be made.

For close interval monitoring, another system involving the use of Shaped Accelerometer Arrays (SAA) provides a much higher density of information at intervals as low as a few minutes. The SAA remains in the hole for the duration of the monitoring. The instrument system is recoverable provided the shear movements are not so large as to shear the hole off. With the high initial cost of the instrument, the SAA has greatest application for monitoring a relatively short interval of a borehole (50-100 m) in circumstances where shear movements are expected to remain small. The SAA system is well suited to identification of shear horizon locations and the timing of initial movements on these shear horizons.

Walsh *et al* (2014) describe the use of an SAA inclinometer system to protect a sensitive sandstone waterfall features from subsidence impacts. This system also provided insights into the mechanics of the processes driving valley closure including those natural processes such as rainfall events and thermal variations (Mills 2014).

Horizontal shear displacements on specific horizon are commonly observed to occur at the following:

- Thin or low strength bedding planes that are continuous over large areas such as tuff bands, coaly horizons, and some clay bands
- The interface between lithological units with high strength or elastic modulus contrasts such as found at the boundaries of igneous sills, volcanic lava flows, massive sedimentary channels, dykes
- Pre-existing faults/joints forming defined blocks or wedges within the rock mass

These surfaces that are frequently the foci of induced slip, define a failure in the rock mass. The accumulation of stress/strain induced by the longwall excavation subsidence event refracts across the boundary until eventual failure. As ground movement due to the nearby mining activity occurs within the overburden strata, they become focussed at these horizons as shear displacements. These

displacements may be progressive and gradual or rapid and sudden as mining progresses. Their characteristics provide insights into the nature of the shear surfaces and whether sudden rock failure is involved or the shear surface is at limiting equilibrium naturally and remobilised as a result of mining.

Survey frequency is dependent on the nature of the events being monitored. If the observation hole is established for a long term monitoring of infrastructure with only small movements anticipated, it may only require quarterly surveying to collect sufficient data to characterise the ground deformation. If more rapid and/or larger deformations are expected, frequent surveying may be required while the borehole remains open.

Plotting and interpretation of recorded data is achieved through use of propriety software supplied by the suppliers of various Inclinometer systems. The can also be processed reasonably easily in a spreadsheet to give greater flexibility.

Independent surveying of the position of the borehole collar is useful to confirm that any reference horizon assumed to remain stationary does indeed remain in the same position throughout the monitoring period. Knowing the borehole position and original orientation allows accurate determinations of the magnitude and orientation of ground movements. Relating the depth of ground movements to the stratigraphy provides useful insights into the key horizons that are mobilised by mining activity.

OBSERVATIONS FROM DEEP INCLINOMETER INSTALLATIONS

In this section, the results of inclinometer monitoring conducted at several sites are presented as an illustration of the effectiveness of inclinometer monitoring in determining horizontal movements around longwall panels.

Figure 7 shows an example from a site where stratigraphy is strongly influencing the deformation behaviour within the overburden strata. The horizon at an RL of 67m is clearly controlling lateral displacements as discrete shear while the 50m thick unit below this horizon is experiencing shear distortion en mass without the formation of discrete shear horizons.

The same inclinometer data can be viewed in a range of different plots to highlight active displacement horizons as well as the magnitude and directions of ground movements at different horizons within the overburden strata. A plot of mean deviation of both axes (Figure 8) again shows shears can be detected and the specific horizon attributed. Plots of vectors can be determined for specific horizons and subsequent surveys complied to reveal movement history at a specific surface (Figure 9).

At another locality, Figure 10, two nearby inclinometer holes E103A and NC533 and coincident stress monitoring holes were used to monitor subsidence over longwall two longwall panel. Figure 11 shows the vectors of horizontal movement observed shear movements on seven primary horizons within the overburden strata above a 15-20m thick conglomerate unit directly overlying the coal seam (Mills *et al* 2015). The vectors of horizontal movement closely align with horizontal stress changes measured at the same site and with the major *in situ* horizontal stress indicating that the movements are primarily associated with horizontal stress relief toward the approaching longwall goaf.

Initial shear movements were observed as soon as the longwall panel started at a distance of 425m ahead of the longwall face. This observation confirms that horizontal stress relief is able to mobilise the overburden strata over large distances

Two inclinometer monitoring holes located some 1,050 m apart at this site show a strong correlation between the elevations of activated shear horizons as indicated in Figure 12. This correlation suggests that the shear horizons may be continuous across large areas.



Figure 7: Example of Cumulative Displacement within a rock mass in stratified geology at an Australian coal mine shown relative to the inclinometer casings orientated A and B axis.



Figure 8: Mean deviation of displacement within a rock mass in stratified geology at an Australian coal mine shown relative to the inclinometer casings orientated A and B axis.











Figure 11: Inclinometer movements measured in the A+ (North - South) and B+ (Ease - West) axis directions (after Mills *et al.* 2015)


Figure 12: Orientation and magnitude of shear movement in relation to principal stress direction in actual measured locations (after Mills et al. 2015).

Figure 12 also indicates the onset of general body shear distortion when the longwall face is within about 30m of the inclinometer hole. This general body shear distortion has quite a different characteristic to the block movement on discrete shear planes observed at greater distances. General body shear distortion occurs within the rock mass itself and is associated with the shear in a vertical direction caused by the downward displacements of the overburden strata that occur immediately behind the longwall face. These shear distortions are much stronger than would be consistent with pure bending of individual stratigraphic units suggesting that interpreting displacements in an around the longwall face purely as a bending phenomenon may not correctly capture the key characteristics of ground behaviour in this area.

CONCLUSION

Measurements of horizontal and oblique displacements within the overburden strata can be made using a range of different systems depending on the particular circumstances. These circumstances include the magnitude of shear movements expected, the depth to the shear horizons of interest and the frequency and detail of the monitoring required.

Inclinometer casing installed in large diameter holes surrounded by granular material to accommodate shear displacements at discrete horizons are found to provide high confidence insights into ground behaviour about longwall panels to depths to 300 m plus.

Coincident use of stress cells and inclinometers provide further insights into the characteristics of ground movement. including have shown two distinct movement styles, firstly a horizontal stress relief horizontal shear, then a tilting and sagging as the excavation is close to undermining a position.

Inclinometer monitoring is providing significant insights into the mechanics of the processes that cause ground movements naturally and in response to longwall mining. Further studies are expected to expand this understanding.

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DEVELOPMENT OF THE ANZI STRAIN CELL FOR THREE DIMENSIONAL IN SITU STRESS DETERMINATIONS IN DEEP EXPLORATION BOREHOLES

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ABSTRACT: The Australia, New Zealand Inflatable (ANZI) strain cell is an instrument used to determine the three dimensional *in situ* stresses with a high level of confidence, through the overcoring method of stress relief. The ANZI cell has been used for over three decades at numerous sites around the world, typically in short inclined boreholes drilled from underground mines. Technical advances during the last decade have seen the ANZI cell deployed and overcored in increasingly deeper surface exploration boreholes. Recent development of a downhole electronic data logger, a wireline enabled drilling system and an instrument deployment system has greatly simplified the process of obtaining three dimensional overcore measurements at depth. This paper describes the ANZI strain cell, its operation and recent development for overcoring in exploration boreholes. The capability to deploy ANZI strain cells in exploration boreholes represents a significant breakthrough for the design of underground mines and underground excavations generally. Being able to obtain high confidence measurements of the *in situ* stresses at the planning stage of any underground construction activity provides the opportunity to take advantage of these stresses. Not only does it become possible to protect key infrastructure by locating it away from areas of stress concentration, advantage can be taken of the major stresses to promote caving through appropriate design.

INTRODUCTION

This paper describes the advances that have enabled overcoring of the Australia, New Zealand Inflatable (ANZI) cell in deep surface exploration boreholes and what a successful test can provide. An overview of the development of the overcoring method of stress relief and of the ANZI cell itself is provided for context. The operation of the instrument and the various stages of testing used to provide confidence in the integrity of each measurement is detailed. Recent developments including wireline enabled drilling techniques used to prepare the pilot hole and a newly developed downhole logger / strain cell assembly used to obtain the strain measurements from which the *in situ* stresses can be estimated at the point of measurement are described.

Before describing any process of "*in situ* stress measurement", the limitations of this terminology should be recognised. The concept of stress is a convenient engineering construct to link displacements and their derivative strain with forces through idealised models of material behaviour Stresses do not actually exist as something that can be measured. In a Continuous, Homogeneous, lsotropic, Linear Elastic (CHILE) material, six independent components of strain change are able to uniquely define a change in a three dimensional stress tensor. Strain change can only be measured by changing the loading conditions acting on a material.

To conduct an "*in situ* stress measurement" requires (1) a change in loading conditions, ideally from *in situ* conditions to conditions of zero stresses, (2) the measurement of sufficient independent strain changes during this process, and (3) an assumption about the material behaviour. To say that the stresses have been "measured" by this relatively involved process is somewhat misleading because the best that is possible is to "estimate" the *in situ* stresses based on imperfect measurements of strain change and an idealised model of the behaviour of rock material. Nevertheless, the term "stress

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measurement" has gained widespread usage in the lexicon and is, at times, more convenient to use, but the limitations of the terminology should be recognised.

OVERCORING METHOD OF STRESS RELIEF

The overcoring method of stress relief is a convenient method for changing the loading conditions from *in situ* stress conditions to conditions of zero stress. By measuring the strain changes during this process in six independent orientations and the full three dimensional *in situ* stress tensor can be determined based on a CHILE model of material behaviour. In practice, there are a variety of influences that are found to complicate this process including drilling induced effects (Mills, *et al.,* 2016), material behaviours that are not captured by the CHILE model (Mills and Gale 2016), and for some types of instrument, the presence of the instrument itself influences the final state of stress in the post-overcored rock.

The overcoring method of stress relief has its beginnings in a technique where rock on a tunnel wall is isolated from the stress field by drilling a series of interconnected holes and the resulting displacements are measured. Lieurance (1933) reports using this technique during investigations for the construction of the Boulder Dam. Olsen (1949) reports using the same technique but with the introduction of strain gauges for the purpose of measuring displacements. The overcoring method of stress relief in boreholes progressed during the 1950s with the development of a variety of different instruments (Leeman 1958, Hast 1958, Obert, *et al.*, 1962).

In the 1960s, the technique developed further so that it became possible to measure *in situ* stresses in three dimensions from one borehole. An analysis presented initially by Leeman and Hayes (1966) and refined for solid inclusion devices by Duncan Fama and Pender (1980) provides a method for estimating the *in situ* stresses from elastic strains measured on the surface of a borehole, most generally using electrical resistance strain gauges bonded directly to the rock or included within a hollow inclusion bonded to the rock. The changes in strain that occur on the borehole wall during the overcoring are assumed to be caused entirely by the response to the change in stress of the rock material.

ANZI STRAIN CELL DEVELOPMENT HISTORY

The original Auckland New Zealand Soft Inclusion (ANZSI) strain cell (Mills and Pender 1986) was developed from 1980 to 1983 at the University of Auckland for the purpose of being able to estimate the three dimensional *in situ* stresses in coal. The primary goal was to reduce the tensile stresses generated at the borehole wall by the presence of the instrument. In soft rocks such as coal, the tensile stresses generated at the borehole wall during overcoring of hollow inclusion instruments can become high enough to cause tensile failure of the rock itself thereby compromising the test. A secondary goal was to develop an instrument where strain gauges bonded directly to the rock could be tested *in situ* prior to overcoring to confirm the behaviour of the rock material was tolerably consistent with a CHILE model.

From this start, the concepts of keeping the strain measurement system as unobtrusive as possible so that the presence of the instrument does not influence the strain measurements, coupled with providing as much redundancy of measurement of strains and material properties as is practical, have guided ongoing development of the instrument.

The ANZSI strain cell was 38 mm in diameter and carried nine strain gauges. The instrument was successfully used to measure *in situ* stresses in coal mines in New Zealand, Australia, and the United Kingdom as well as at several hard rock civil sites in New Zealand. In 1990, the instrument underwent a significant upgrade and a name change. The diameter was increased to 56 mm and manufactured on a hollow, tubular body. The number of strain gauges on each instrument was increased to 18, and the name was changed to ANZI (Australia New Zealand Inflatable) strain cell reflecting the instruments combined development history and essential mode of operation.

Since the first beginnings in 1980, the ANZI strain cell has continued to be developed through incremental improvements that have greatly increased its capability over time. Most recently, successful deployment and overcoring in a surface drilled exploration at 850 m depth below the ground surface was achieved.

The ANZI strain cell has been developed with focus on simplicity of operation and providing high levels of redundancy to give a sense of the confidence that can be placed in each individual point measurement. Available analysis techniques for converting measured strains to stresses are limited by assumptions that the material is linear, elastic, isotropic and homogeneous. However, many rocks in which overcore tests are conducted are not ideal materials. These rocks are commonly not linear, elastic, isotropic, or homogeneous. Furthermore, the material properties of some softer rocks are commonly observed to change with a change in stress.

Recognising that the calculation of stresses from strains is imperfect, the key to obtaining value from the measurement is gaining a sense of the confidence that can be placed in each measurement and how well the rock properties can be approximated as an ideal material. Experience show not all measurements aimed at determining the *in situ* stress field or changes in stress are reliable. However, having a basis to differentiate those that are high confidence from those that are not is invaluable when developing an overall understanding of the stress environment and the rock behaviour within that environment. The design of the ANZI strain cell is focused on providing systems to allow the confidence in each point measurement to be assessed.

OPERATION OF THE ANZI STRAIN CELL

The ANZI strain cell is a strain measuring instrument that uses the overcoring method of stress relief to allow the *in situ* stresses to be estimated from the strains measured on variously oriented strain gauges. The gauges are bonded directly to the rock on the wall of a borehole. Figure 1 shows a photograph of the 48 mm diameter version of the instrument. The instrument has an inflatable membrane of soft rubber-like material with multiple strain gauges exposed on its outer surface. Eighteen electrical resistance strain gauges at various orientations are mounted flush on the outside surface of the membrane. When the membrane is inflated during installation, the electrical resistance strain gauges become cemented to the borehole wall allowing direct measurement of strain changes in the rock. The wiring of the strain gauges is embedded in the membrane so that the instrument is waterproof. Reference gauges on the instrument that do not change, instrument orientation, inflation pressure, water pressure in the hole, and temperature are also now routinely monitored.

300 -- 0



b) Deployment assembly, downhole logger and ANZI strain cell immediately prior to installation.

a) 48mm diameter ANZI strain cell.

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c) Downhole logger, recovered overcore and compass module.

There are six stages in the standard ANZI strain cell test procedure: preparation of the hole, installation, *in situ* pressure test, overcoring stress relief, biaxial pressure test, and laboratory testing of the core recovered from the pilot hole.

Borehole preparation

A borehole is drilled to the measurement location using standard drilling procedures. This borehole is now most commonly an HQ size (96mm diameter) borehole, but a range of other options are available and have been used. The end of the hole is prepared so that the core stub is removed and a centralising conical indentation is formed. A smaller diameter pilot hole is then drilled concentrically from the end of the larger diameter hole, typically for a distance of about 1m. The core from this pilot hole is inspected to determine an optimum test interval. The core is retained for material testing in the laboratory.

Installation

To install the ANZI strain cell, the outer surface of the instrument is coated with custom designed epoxy cement. The instrument is then installed into the pilot hole at the target depth. Pressure is applied internally to the membrane causing the strain gauges to be pressed directly against the borehole wall. Most of the epoxy cement coating is extruded away from the strain gauges and the membrane leaving only a very thin 0.3-0.5 mm thick layer between the gauges and the rock. When the cement has cured, typically 3-4 hours depending on rock temperature, the strain gauges are bonded directly to the rock.

In situ pressure test

Once the cement has cured, the internal pressure is varied incrementally to conduct a pressure test using the instrument as a dilatometer or pressuremeter. The pressure changes in this test are kept relatively low to avoid disturbing the *in situ* stress field. The strain changes measured (typically 20-200 microstrain) are sufficient to confirm the correct operation of each individual stain gauge, provide a measure of the *in situ* properties of the host rock before it is disturbed by drilling, and, under some circumstances, provide independent confirmation of the *in situ* stress direction (Mills and Gale 2016).

The pressurised length of the ANZI strain cell membrane is designed to be four times the diameter of the borehole so as to generate near plane strain conditions during the *in situ* pressure test (Laier *et al* 1975). The increased length of the instrument also improves the length of overcore recovered in low strength or highly jointed rock.

Overcoring

The ANZI strain cell overcoring operation is conducted in much the same way as for other instruments that use the overcoring stress relief method. Direct bonding of the strain gauges onto the surface of the borehole means that the diameter of the overcore need only be slightly greater (10-20 mm) than the diameter of the pilot hole and instrument for the result to be valid. The zero stress state of the final overcore means the overcore does not need to remain completely intact or maintain a regular geometry for the result to be valid. These characteristics significantly extend the range of rock types and drilling environments in which the instrument can be used.

The configuration of strain gauges carried on the instrument can be varied to suit rock conditions. Typically, 5mm long gauges oriented in rosettes of three gauges each (0°, 45° and 90° to the axis of the borehole) are used. The 5mm long gauges minimise the strain averaging effect of longer gauges that can affect results in some stress fields. The gauges oriented at 0° and 90° orientations facilitate field interpretation of results.

The six rosettes of three gauges each are oriented at 60° intervals around the circumference of the cell to improve statistical confidence in the *in situ* stress measured (Gray and Toews 1974). Each rosette has one gauge oriented in a circumferential direction. Multiple gauges are oriented in an axial direction. Eighteen gauges gives 12 degrees of redundancy and two or more independent measurements of many of the individual strain components. For instance, the three sets of directly opposite circumferential gauges independently measure the same strain value.

With the downhole logging system, strain, pressure and temperature readings are recorded every few seconds commencing before the instrument is deployed until it is recovered, leading to a high data density. Figure 2 shows the strain changes associated with stress relief measured during overcoring for each of the eighteen strain gauges. The general form of the overcoring strain changes can be used as a basis to identify rosettes of strain gauges that may not be behaving in a manner consistent with a strong result.

In situ stresses are calculated from the measured strains using the technique described by Leeman and Hayes (1996) and variously enhanced by others. A minor correction can be made during analysis to include the effect of the 0.3-0.5 mm thick epoxy cement layer formed between the membrane and the rock using the analysis described by Duncan-Fama and Pender (1980), but the effects of this correction are slight. For all practical purposes, the strain gauges can be considered bonded directly to the borehole wall.

The membrane material has a modulus of elasticity of only a few mega Pascals and so is soft enough to be ignored in the analyses. Significantly, the tensile stresses generated at the rock/instrument interface during overcoring are too low to overload either the epoxy cement bond strength or the tensile strength of the rock for most rock materials ensuring the integrity of the overcore measurements is maintained in a broad range of difficult drilling conditions.



Figure 2: Strain changes during stress relief from overcoring measured at 3 second logging intervals, at 850m depth in an exploration borehole.

Biaxial pressure test

A biaxial pressure test is conducted after the overcore is recovered. In this test external pressure is applied to the rock cylinder so the elastic modulus and Poisson's ratio of the rock material can be estimated. The overcored rock annulus is incrementally pressurised in a biaxial cell that applies radial pressure to the outside of the overcore. The biaxial test provides measurement of the elastic modulus

and Poisson's ratio at a range of different pressures and is useful as an indicator of the sensitivity of the rock to modulus variations with pressure.

Laboratory testing

A laboratory test of the core recovered from the location of the measurement is tested in a multi-stage uniaxial compression test. Axial and circumferential strain gauges and the load/displacement records of the compression test all the elastic properties of the rock to be estimated during three or more load/unload cycles up to failure in uniaxial compression.

Assessment of elastic properties

The elastic properties of the rock mass are determined in three separate tests;

- 1. the *in situ* pressure test conducted prior to overcoring
- 2. the biaxial pressure test conducted after overcoring, and
- 3. laboratory tests on core recovered from the pilot hole.

These three essentially independent measurements are conducted on the rock in various stages of the overcoring process and therefore at various levels of stress.

These different conditions provide insight into the rock behaviour and the impact of drilling on the rock as it is unloaded and recovered from the hole. In an ideal, homogeneous, linear, elastic, isotropic material, all three tests would indicate the same values of elastic properties. However, variations are commonly observed, and these variations have provided useful insights into a range of factors that affect the material behaviour of these rocks.

OVERCORING IN EXPLORATION BOREHOLES

The ANZI strain cell was first used in a surface exploration hole in 2009. The instrument is now routinely overcored at depths ranging 200-400 m, with two successful results achieved recently beyond 800 m depth. The benefits of understanding the *in situ* stress field during exploration and the design phase of an underground excavation are significant. With relatively little effort once the *in situ* stress orientations are known, underground excavations can be laid out to minimise stress concentrations on key infrastructure, reduce the investment in reinforcement, and yet still take advantage of elevated *in situ* stresses to fracture rock and induce caving without the need for blasting. The potential to realise these benefits by being able to obtain high confidence estimates of the three dimensional *in situ* stresses has driven the development of the ANZI strain cell for use in exploration holes. A number of key challenges were overcome to enable successful stress measurements at depths beyond 800 m. These were met incrementally as the ANZI strain cell was deployed at progressively greater depths over the past eight years.

Overview of key developments

Initially the same drilling equipment and installation techniques used in underground overcoring were used in surface boreholes. Installation rods routinely used underground were used in shallow installations down to about 150 m but were found to be difficult to use at depths greater than about 50 m. Solid installation rods were replaced with a two cable system deployed on a mechanised cable drum through the drill rods. The first cable was connected to the instrument and monitored at the surface for the duration of the overcoring. The second cable was used to pressurise the instrument during installation and the pressure test.

The 56 mm diameter version of the ANZI strain cell was used initially but this required specialist core barrels because the standard HQ core size is 61 mm in diameter. Several core barrel configurations were developed and trialled but the challenges of swapping out core barrels became a significant

impediment to the ease of conducting measurements. A new 48 mm diameter version of the ANZI strain cell was developed to allow overcoring with standard HQ3 equipment to streamline the process.

In order to eliminate rod tripping, a wireline deployed downhole drilling system was implemented. This system is based on a standard casing advancer system that provides for a wireline deployable downhole drive. The end of the hole is able to be shaped and a 48 mm diameter pilot hole drilled using an LTK48 core barrel without needing to trip the rods and replace the HQ barrel. A self-contained downhole logger module and new deployment system were designed to eliminate the need to run a data cable from the instrument to the surface. This final piece of the system enables the ANZI strain cell to be deployed routinely to much greater depths than was previously possible.

Overcoring using custom coring barrels

In 2009, several ANZI strain cells were overcored in HQ surface exploration boreholes at depths of generally less than 30 m. Multiple rod trips were required to change the drilling bit/core barrel to prepare the pilot hole. A 58 mm diameter ANZI cell was installed using PVC conduit, with the data cable and inflation line running through the centre. After the epoxy glue used to bond the strain cell to the pilot hole had cured, the PVC conduit and inflation line were removed and a custom built single tube core barrel with a 76 mm ID shoe bit was run in. The process required manually feeding the data cable through each additional drill rod added. The data cable was run through a modified water swivel on the drilling rig rotation drive unit. Once the stresscell had been overcored and the core had detached, the instrument with the overcored rock attached was recovered through the HQ rods via the data cable.

For a two year period during 2010-2011, approximately 12 overcores were conducted at depths ranging 30-50 m using this system. The internal diameter of the core barrel was increased, to aid drilling circulation and reduce the fluid pressure acting on the stresscell during overcoring. A rotation sensor with an audible alarm was fitted to the ANZI strain cell to indicate when the overcore had detached, so rotation could be immediately stopped to prevent cable twist. In 2012, a similar system using PVC installation conduit was employed to achieve a successful measurement at 150 m. Due to the depth increase and inability to confirm a correct landing from surface using standard technique, a mechanical stopper was fixed to the back of the instrument assembly to land at the top of the pilot hole. The stopper was adjustable and fixed at a position that placed the strain gauges at an optimum location in the pilot hole. From this time, the custom overcore barrel and drill string were run to the bottom of the hole prior to the installation of the instrument. This approach eliminated the requirement to feed the data cable through each drill rod as it was added. The complete drill, install, and overcore operation at this depth took nearly two days owing mainly to the rod tripping time required to prepare the pilot hole.

Cable winch

A project was commissioned in 2013 to conduct an overcore at 300 m depth. Installation using PVC conduit was not possible at this depth due to the weight of the system. A hydraulically powered cable drum winch was designed and built specifically for the purpose of installing the instrument. The instrument could be deployed on the data cable via a pulley system on the drill rig mast and the inflation line could spool into the hole in parallel from a separate cable drum. An electronic compass was added to the strain cell to provide orientation. Tripping the rods in and out of the hole to change out the core barrels was found to introduce fines into the pilot hole. A longer pilot hole was used to accommodate the fines and a landing sensor was added to the stresscell stopper to provide confirmation the instrument has fully entered the pilot hole, prior to applying inflation pressure from surface. These strategies were only partly successful and the rod tripping was still excessively time consuming.

Small diameter instrument

A new pilot hole preparation technique was required to allow the pilot hole to be drilled without the need to trip rods and at the same time allow drill rods to act as effective casing to prevent fines entering the pilot hole. For this system to be truly effective, the drilling system would be capable of being integrated into routine drilling operations using the 61 mm ID HQ core bit for the overcoring, to eliminate the need for a single rod trip. A 48 mm diameter ANZI strain cell was developed to work in a pilot hole that could be prepared using an LTK48 core barrel and then overcored using a standard HQ3 bit. A 48 mm ANZI strain cell was successfully installed through the rods into the 48 mm diameter pilot hole using the two cable mechanical winch deployment system and then overcored with the conventional HQ bit without the inner tube in place. The overcore was recovered after overcoring using the stresscell data cable by pulling it up through the rods.

Wireline deployed drilling system

In 2014, a wireline deployed downhole drilling system was used successfully to prepare a 48 mm pilot hole for an overcore measurement at 160 m depth. The system requires the addition of a short casing advancer driver sub located between the locking coupling of the core barrel and the first drill rod. The driver sub can be installed at any time because it does not interfere with routine drilling operations. The downhole drilling assembly is pumped down the rods in a fashion similar to the inner tube, landing in the driver sub and protruding out through the HQ bit. The HQ rod string is raised off bottom prior to deploying the downhole drilling assembly sufficiently that the protruding bit does not contact the end of the hole. The downhole drive drilling fluid to the downhole drive bit face. Two deployments of the downhole drive are required to prepare the pilot hole. The first is to grind out the HQ core stub leaving a conical indentation in the centre of the HQ hole to centralise the barrel. The second is to drill the pilot hole using the LTK48 core barrel. The downhole drilling assembly is retrieved on the overshot. In 2014/15 six successful overcore stress measurements using the downhole drive drilling system to prepare the pilot holes and 48 mm ANZI strain cells were conducted at depths between 250 m and 350 m.

Development of a downhole logger system

A decrease in gauge sensitivity owing to increasing cable lengths and the challenges of handling long cables provided the impetus to investigate the feasibility of a self-contained data logger that could be fixed to the back of the stresscell. The data logger presented a technical challenge but significantly increases the capability of the stress measurement system. The downhole logger has the following benefits over a cable system with measurement at the surface:

- increases accuracy, stability and frequency of strain readings achievable because of shorter data cable lengths
- permits internal and external pressure and temperature readings without adding to cable weight
- eliminates the need to use the hydraulic cable winch system for deployment
- reduces amount of equipment required thus allowing air-freighting of gear
- reduces labour requirements from a two person operation to a one person operation
- reduction in manual handling and elimination of all significant hazards
- overall reduction in drilling rig downtime and associated cost savings to the client

In December 2015 a prototype logger housing was constructed. This housing is designed to withstand a maximum working pressure of 10 MPa (or a 1000 m deployment). The downhole logger electronics were ready for use in June 2016 after approximately six months of design, manufacture, and testing.

The downhole logger system also required a complete re-design of the instrument deployment system. This process took some three months to complete. The deployment system consists of two separate modules that are deployed together but recovered separately at two different stages in the stress measurement process. The full assembly consists of an upper landing module and a lower logger module. The complete assembly is lowered on the overshot with the dry-release in place. Once the assembly reaches the water level in the hole, it is released and allowed to float down the inside of the drill pipe. The assembly seats in the core barrel landing ring and cell inflation is achieved through pressurisation of the drill pipe from surface through a series of downhole valves and seals.

Once the epoxy cement has cured and the *in situ* pressure test has been conducted, the upper module detaches from the lower module and the upper assembly is recovered using the overshot on the wireline. The inner tube is then run in and seated with the downhole logger inside. The ANZI strain cell is then ready to be overcored. In June 2016, after six months of laboratory and field trials, the downhole logger and deployment systems were used for the first time. Three successful overcore stress measurements were conducted at depths between 200 m and 300 m using the wireline drilling system and the modular deployment system and downhole logger.

Cement cure time optimisation

Testing of the cement cure times allowed the total elapsed time between installation and the commencement of the pressure test to be reduced to about 4-5 hrs depending on ambient rock temperature in the hole. This discovery makes same day overcoring possible. One of the three overcores described above was conducted on the same day the instrument was installed.

Overcores to 850 m

In November 2016, an opportunity came up to undertake overcore stress measurements at depths greater than 400 m in an inclined borehole. Two main challenges needed to be overcore:

- increased descent rate to prevent the epoxy cement from curing prematurely
- centralising of the deployment system in HQ rods to facilitate landing in inclined boreholes.

Laboratory tests to confirm the epoxy cement curing time indicated that the stresscell would need to descend at a rate of twice normal descent rates to be effective at 500 m+ depth. A rapid descent back-end containing a series of valves was developed to increase descent rate and reduce travel time. A field test was conducted and confirmed a two-fold increase in descent rate was confirmed as was the operation of the internal valving system. After some challenges with getting the assembly to land properly and some redesign of the deployment system to include a centralising tube and release mechanism, successful overcore measurements were made at 547 m deep in one orebody and again at 811 m and 850 m in a second orebody. Each stress measurement was able to be conducted in a single 10 hr shift with a return to normal drilling operations immediately after.

Capability improvements

The capability to deploy ANZI strain cells in exploration boreholes represents a significant breakthrough for the design of underground mines and underground excavations generally. Being able to obtain high confidence measurements of the *in situ* stresses at the planning stage provides the opportunity to take advantage of these stresses. Not only does it become possible to protect key infrastructure by locating it away from areas of stress concentration, advantage can be taken of the major stresses to promote caving.

CONCLUSION

The capability to deploy ANZI strain cells in exploration boreholes represents a significant breakthrough for the design of underground mines and underground excavations generally. Being able to obtain high confidence measurements of the three dimensional *in situ* stresses at the planning

stage of any underground construction activity provides the opportunity to take advantage of these stresses. Not only does it become possible to protect key infrastructure by locating it away from areas of stress concentration, advantage can be taken of the major stresses to promote caving through appropriate design.

The ANZI strain cell has a range of operational features and analytical simplicities that have enabled *in situ* stresses to be successfully determined and stress changes to be successfully monitored in a wide range of rock types and applications over the last three decades.

The high levels of redundancy in both the instrument and the measurement technique are designed to provide an indication of the confidence that can be placed in each result and to enhance the understanding of material behaviour at the point of measurement and ground behaviour at the site more generally.

The development history of the instrument has been described in this paper together with the key steps that enable high confidence measurements to now be made in exploration boreholes at depths in excess of 800m. These measurements are possible to conduct within a few hours allowing overcore measurements for the determination of the full three dimensional *in situ* stress field to be made a routine component of exploration activities.

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FUNDAMENTAL PRINCIPLES OF AN EFFECTIVE REINFORCING ROOF BOLTING STRATEGY IN HORIZONTALLY LAYERED ROOF STRATA AND THREE AREAS OF POTENTIAL IMPROVEMENT

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ABSTRACT: It is arguable that the development of reinforcing roof bolting systems has largely stagnated in recent times primarily due to the prevailing industry view that few, if any further improvements can be made to the current state of the art. On the contrary, this paper will contend that reinforcing roof bolting systems can be further refined by considering both the specific manner by which horizontally bedded roof strata loses its natural self-supporting ability and the specific means by which reinforcing roof bolts act to promote or retain this natural self-supporting ability.

The Australian coal industry's seeming insistence on minimising bolt-hole diameter to maximise load transfer and targeting full-encapsulation by any means has led to a significant, albeit unintended consequence in terms of overall roof bolting effectiveness, namely the promotion of increased resinpressures during bolt installation and the associated potential for the opening up of bedding planes that may otherwise remain closed during the bolt installation process. Given that the natural selfsupporting ability of roof strata is strongly linked to whether bedding planes remain open or closed, it stands to reason that minimising resin pressures should be of significant benefit. Three issues are primarily focused on three key issues that relate directly to the function of the roof bolting system itself, namely: (i) the importance of proper resin mixing in the context of maximising load transfer strength and stiffness, (ii) the importance of minimising resin pressures developed during bolt installation and (iii) the importance of maximising the effectiveness of the available bolt pre-tension. The logic being that if: the reliability of resin mixing with varying hole diameter is substantially improved, if resin pressures generated during bolt installation are substantially reduced, if the length of the bolted interval directly influenced by high resin pressures generated during bolt installation is substantially reduced, and if roof bolt pre-tension levels are increased, why wouldn't individual roof bolt effectiveness and thus roof reinforcement improve?

The potential benefits to the mining industry of improving the individual effectiveness of each installed roof bolt, even by relatively small incremental amounts, should be of interest to all mine operators and is an important topic for discussion amongst the mining community.

INTRODUCTION

The installation of primary roof bolting as part of the roadway development operation is the most obvious "pro-active" strata control process that is available to mining operations. The extent by which primary roof support is installed suitably close to the development face and is geotechnically fit for purpose, sets in place the conditions, good or bad, that will ultimately determine such operational outcomes as triggering of the TARP, subsequent roof deterioration and/or instability and the need or not for high density and expensive secondary or remedial support measures. However in an overall industry context, the effectiveness of primary roof support has received far less attention in more recent times as compared to such areas as geotechnical characterisation, geotechnical design and operational strata management. This paper re-visits the subject area by examining three technical

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areas whereby substantial improvements can potentially be made based on the published findings of a range of research studies and specific testing data.

The development of reinforcing roof bolting in underground coal mining, which is the mainstay of safe and efficient mining, has reached a point whereby the design and set-up of the bolting system can be further refined by considering the manner by which the roof strata loses its own self-supporting ability, reinforcement being the promotion or retention of natural self-supporting ability within the host rock mass. By understanding both the de-stabilising mechanisms within the roof strata itself and the various influences that primary roof bolts have on those mechanisms, the set-up of each installed roof bolt can be optimised to achieve the highest level of individual roof bolt effectiveness. The potential benefits to the mining industry of such an outcome as compared to using less effective bolting systems should be self-evident and requires no further discussion.

Without digressing into a detailed history of roof bolting development over the past 40 to 50 years, it is contended that an optimum roof bolting system needs to incorporate measures that address at least eight fundamental principles of roof reinforcement:

- 1. The position of bolt installation with respect to the development face (i.e. cut-out distance)
- 2. Use of an appropriate bolt length and a geotechnically suitable bolt pattern
- 3. Minimising resin pressure developed during bolt installation in an attempt to minimise any adverse effects on the roof strata within the bolted interval
- 4. Ensuring proper resin mixing when generating the bond between the bolt and surrounding strata
- 5. Utilising a resin system with properties that act to promote increased load transfer strength and most importantly, load transfer stiffness
- 6. Maximising the effectiveness of the bolt pre-tension generated via nut tightening at bolt installation
- 7. Protecting persons in the mine from any roof material that may detach between bolts
- 8. Applying an on-going operational process to both correctly install ground support as well as manage and control the inherent uncertainties in the stabilisation of a naturally formed engineering material

Applying these eight fundamental principles leads to various insights as to how a reinforcing roof bolting strategy can be best-optimised, this paper considers in varying detail three issues that relate directly to the set-up of the roof bolting system itself, namely:

- (i) proper resin mixing in the context of maximising load transfer strength and stiffness
- (ii) the importance of minimising resin pressures developed during bolt installation
- (iii) maximising the magnitude and effectiveness of the bolt pre-tension developed at installation

The discussion around each of these aspects will be based on an analysis of how the primary source of self-supporting ability in layered roof strata is retained, how such natural roof stability is lost and the various interactions between installed roof bolts and the occurrence of de-stabilising mechanisms.

SELF-SUPPORTING ABILITY IN LAYERED ROOF STRATA

Figure 1 illustrates a simplified representation of the three fundamental sources of roadway roof stability in a layered and jointed rock mass under the action of some level of horizontal stress (UNSW 2010). The three stabilising mechanisms are (i) cohesion between bedding planes, (ii) horizontal stress acting to prevent shear slip along sub-vertical jointing within the roof strata and (iii) some form of "suspension" type support to hold-up a roof mass that does not contain the natural stabilising benefits of (i) and (ii). Without at least one of these mechanisms in place, a major roadway roof fall is an inevitable consequence.

On the assumption that utilsing a suspension roof control strategy is not a preferred approach in high production underground coal mining, the critical importance of preventing horizontal separations occurring within the roof strata is self-evident. Firstly the opening up of bedding planes directly causes the loss of bedding plane cohesion (stabilising mechanism (i)) and if sufficient closely-spaced bedding planes open up, it can lead to the *en masse* buckling of the roof strata and an associated reduction in horizontal stress levels (stabilising mechanism (ii)), as explained in detail in Colwell and Frith (2010 and 2012).



Figure 1: Schematic representation of the three sources of roadway roof stability (UNSW, 2010)

A real-world demonstration as to the significance of bedding plane condition to the self-supporting ability of layered roof strata is found in Figure 2, which is derived from the US extended cut database (Mark, 1999) used to evaluate roadway roof stability without roof bolts installed. The two-axes represent the varying compressive strength or UCS of the roof material (x-axis) and bedding cohesion within the roof (y-axis) for each of the database case histories, the estimation of the latter being part of the underground method for determining the Coal Mine Roof Rating (CMRR) as was used in the Mark,(1999) study.

The key feature of Figure 2 is the line or boundary that best separates the "*always stable*" from the "*never stable*" cases, as this provides an indication of the relative importance of the UCS of the roof material, as compared to bedding plane cohesion within the roof, to either the retention or loss of natural roof stability (self-supporting ability).



Figure 2: US extended cut stability database assessed for both UCS and bedding cohesion (data sourced from Mark, 1999)

A "by-eye" discriminant line/zone (in blue) is shown in Figure 2 and it is clear that it only "discriminates" between stable and unstable cases according to varying bedding plane cohesion – i.e. lower cohesion (greater than \approx 3) is linked to "never stable" cases and higher cohesion (less than \approx 3)

to "always stable" cases. Furthermore the "always stable" cases cover the full UCS range from low to high meaning that UCS is not a reliable predictor of natural roof stability (as described in detail by Frith and Colwell, 2006).

The point to be made from Figure 2 is that the roof almost certainly loses its natural stability or selfsupporting ability in line with the opening of bedding planes (termed "delamination"). It logically follows that the higher the level of delamination within the bolted interval, the higher the level of installed roof support that will be required to maintain adequate levels of roof stability (all other factors being equal). Bolted roof reinforcement should therefore be primarily focused on preventing bedding planes opening-up within the bolted interval, with minimising the degree of bedding separation once they are open being a second-order, albeit still relevant, consideration.

The set-up of a reinforcing roof bolting system will be further considered based on the concept that retaining the self-supporting ability of the roof strata is primarily based around preventing bedding planes opening up within the bolted interval, but accepting that if they do open up it is nonetheless beneficial to minimise the level of separation that occurs.

RESIN MIXING AND MAXIMISING LOAD TRANSFER PROPERTIES

The entire subject of maximising the load transfer properties of resin-encapsulated roof bolts has been widely researched based largely on both in situ short encapsulation pull-tests and laboratory based pull tests and/or push tests. The general outcome of this work, in Australia at least, was that in order to maximise load-transfer strength and stiffness, the roof bolting system should be fully encapsulated and that the annulus between the bolt and surrounding strata should be as small as possible. When considered in isolation, the logic behind maximising load transfer makes logical sense and remains the current norm in the Australian coal mining industry. However, work from New Zealand, published by Campbell and Mould (2003) as well as Pastars and McGregor (2005), found that there was a fundamental problem with the 15:1 ratio (mastic to catalyst) resins systems that were almost universally used in the Australian coal industry at that time, namely they were prone to poor resin-mixing towards the top end of the bolt and also reduced load-transfer due to "gloving" of the bolt via large pieces of plastic film corrupting the integrity of the resin bond between the bolt and the surrounding strata. A combination of both poor resin mixing and gloving was found to give very low load transfer properties, the existence of which is "hidden" from view during normal mining operations and cannot be easily audited or directly monitored. No practical solution was found from this work to overcome these problems using an industry standard 15:1 resin system.

The idea that load transfer is maximised by minimising the annulus between the bolt and surrounding strata was brought into question by the work of Hagan and Weckert (2004). Lab-based pull-testing using a "mix and pour" resin system rather than a "spun through" resin as used in actual bolt installations, showed no discernible difference in load transfer properties, neither strength nor stiffness, for hole diameter variations between 28 mm and 30 mm (see Figure 3). This finding is directly contrary to what had been published in the past (see Figure 4 after Fabjanczyk and Tarrant, 1992), which in hindsight has almost certainly driven the industry practice of using the smallest possible bolt hole diameter (as low as 26.5 mm).



Figure 3: Load v displacement data for roof bolt pull-testing using Mix and Pour resin (Hagan and Weckert, 2004)



Figure 4: Effect of hole diameter on load transfer (from Fabjanczyk and Tarrant, 1992)

The logical conclusion that can be drawn from these independent areas of research is that the effectiveness of resin mixing within a 15:1 resin system is highly dependent upon minimising the hole diameter, this then leading to the common finding with *in situ* short encapsulation pull test studies, that load transfer increases as a direct function of decreasing hole diameter.



Figure 5: Schematic illustration of the various roof bolting system components during bolt installation and resin mixing

This theory was put to the test as part of ACARP Project C21023 (McTyer, 2015) whereby it was conclusively shown that resin mixing effectiveness for 15:1 resins substantially reduced with an increasing hole diameter from 28 mm to 30 mm, whereas there was little or no such reduction when using a US-style 2:1 resin system (the interested reader is directed to the full project report for the detailed findings). Figure 5 illustrates why this is likely to be the case via a simple cross-section of the bolt hole and the relative cross-section areas and locations of the bolt, mastic and catalyst sections.

From this it is inevitably concluded that the Australian coal industry's seeming insistence on minimising bolt hole diameter to maximise load transfer, is almost certainly a direct function of resin mixing limitations of 15:1 resin systems according to increasing hole diameter. As will become apparent in the next section of the paper, the use of the smallest possible roof bolt hole has a significant, albeit unintended consequence in terms of overall roof bolting effectiveness, namely the promotion of increased resin-pressures during bolt installation, the associated potential for the opening up of bedding planes near the bolt hole and resin losses into the roof strata as a direct consequence.

RESIN PRESSURE DEVELOPMENT DURING BOLT INSTALLATION

The entire issue of resin pressure development during bolt installation and its significance for roof reinforcement can be considered based on a combination of:

- resin pressure measurements made during bolt installations by several researchers,
- a theoretical treatment of the key parameters that influence the development of resin pressures,
- common fracture patterns observed within the bolted interval,
- Griffith Crack Theory, and
- published test data showing the clear links between less than theoretical bolt encapsulation being achieved with various changes made to the bolting set-up (e.g. resin volume used and varying hole diameter).

Comments will also be included pertaining to a recently published technical paper (Purcell *et al*, 2016) which purported to significantly diminish the significance of resin pressures generated during bolt installation to roadway roof stability using a series of technical arguments and roof bolt installation testing results that are judged to contain several fundamental oversights.

An early indication of the significance of resin pressures developed during roof bolt installation is found in Pettibone (1987) whereby the fracturing of 31 MPa concrete blocks is reported as a direct

result of roof bolt installation. The significance to roadway roof stability was not directly considered at this time, but the link was made between roof bolt installation and potential fracturing of the host material.

Compton and Oyler (2005) reported resin pressure measurements (Figure 6) during installation, albeit without spinning during installation, of a 1.2 m long x 5/8 inch roof bolt in a 1 inch (25.4 mm) steel pipe resulting in a 4.7 mm thick annulus (which is equivalent to a 21.7 mm core diameter roof bolt as used in Australia being installed in a 31.1 mm diameter hole). At an insertion rate of 128 mm/second (equivalent to a 1.2 m bolt being fully inserted to the back of the hole in 9.5 seconds – see Figure 6), the maximum pressure measured is in the order of 6000 psi (or 41.4 MPa) at the top of the hole, which is of a greater magnitude than the setting pressure for longwall shields.

Whilst the bolt installation method used is fundamentally different from that used in Australia whereby spinning of the resin starts at the base rather than top of the hole, the results reported by Compton and Oyler (2005) provide an indication of the potential resin pressure magnitudes that can be generated (they report a maximum measured pressure of 68 MPa) using the available power of a hydraulic roof bolting rig.



Figure 6: Hydraulic pressures generated at four locations in a 1 inch pipe during roof bolt installation (5/8 inch bolt) – Compton and Oyler (2005)

A combination of very high resin pressures being measured during roof bolt installation and the observation of blocks of strong host material being fractured as a direct consequence, leads to the inevitable conclusion that resin pressures are indeed significant and have the potential ability to "hydro-fracture" the roof strata, particularly in the upper portion of the bolt where the highest pressures are generated during bolt installation. The logical questions that follow from this recognition are:

- (i) Does the action of such resin pressure potentially detract from overall roof stability?
- (ii) If the answer to (i) is "yes", which roof types are most affected by such action?
- (iii) What are the controls of resin pressure generation and can they be implemented in practice to minimise or even eliminate any negative impact on roof stability whilst not compromising other key reinforcing aspects of the bolting system?

The answer to (i) can be found by reference to observed or inferred fracture patterns within the bolted interval of mine roadways combined with the previously justified statement that the self-supporting ability of roof strata is primarily retained by preventing or at least minimising loss of bedding plane cohesion within the roof strata.

Figure 7 shows a roof fall cavity (with extruded resin "pancakes" clearly evident at the top of the fall profile) and an associated borescope observation plot of the location and distribution of open fractures in the roof strata. The installed roof bolts were 2.1 m long and were designed to be fully encapsulated using 1200 mm long resin cartridges in a hole drilled with a 27 mm bit. The presence of more intense roof fracturing in the upper section of the bolted interval is clearly evident in the borescope data. The geotechnical reasoning for such fracturing is not obvious, however the action of resin pressure forcing resin into the roof strata and so opening up bedding planes can be reasonably inferred from the available evidence.

Figure 8 shows a series of sonic-probe roof extensometers from an ACARP Project field trial at Tower Colliery with the condition of the upper section of the bolted interval being consistently fractured as compared to the overlying strata, to the extent that the bolt length in use can be reasonably identified from the extensometer plots.

In contrast, Figure 9 contains a series of aged sonic probe roof extensometer plots linked to roof bolt installations via hand-held compressed air roof bolters, which logically have a far lower ability to generate high resin pressure during bolt installation. The change in roof fracturing in the upper section of the bolted interval as compared to Figure 8 is self-evident, the potential implication being that far less roof fracturing is present in the section of the bolted interval where maximum resin pressure is generated.



Figure 7: Roof fall cavity and bolted interval fracture pattern from an adjacent borescope hole



Figure 8: Sonic probe extensometer data from ACARP project C3032 (ACIRL 1995)



Figure 9: Sonic probe extensometer data related to hand-held pneumatic roof bolters

In terms of the answer to question (i), if the natural self-supporting ability of roof strata is strongly linked to whether bedding planes remain open or closed (Figure 2) but resin pressures during bolt installation act to initiate and/or aggravate the propagation of open fractures within the upper section of the bolted interval via pumping resin along such fractures, then it stands to reason that roadway roof stability is negatively impacted as compared to such fracture development not occurring.

In terms of which roof or strata types are most likely or easily affected by the influence of resin pressures, this can be considered by applying the various principles of hydro-fracturing, as was used by Purcell *et al* (2016) in their commentary on the subject.

Fracture development from a borehole is based on the hydraulic pressure being applied being able to overcome two distinct resistive forces to fracture development -1) The stress acting across the plane of fracture development; 2) The cohesion (intact tensile strength) of the rock mass again across the fracture plane. As a result of 1, fracture development from a vertical borehole typically propagates perpendicular to the lower of the two relevant horizontal stress magnitudes as this represents the lowest possible resistance of the *in situ* stresses to fracture propagation. The relevant equation governing fracture propagation as quoted by Purcell *et al*, (2016) from Amadei and Stephansson (1997) is as follows:

$$\sigma_1 = 3\sigma_2 + S - P_1 - P_0 = 3\sigma_2 - P_r$$

where: $\sigma_1 = major principal stress$

- σ_2 = intermediate principal stress
- S = tensile strength of the rock perpendicular to the fracture direction
- P_i = crack initiation pressure
- $P_o = pore pressure$
- P_r = crack re-opening pressure

Using Equation 1 (which was specifically developed for predicting crack initiation and re-opening pressures for vertical cracks in a vertical borehole as part of hydro-fracturing stress measurement), Purcell *et al* (2016) apply a basic model for the *in situ* major and minor horizontal stresses in underground coal mines which in combination with what they state to be a moderate tensile strength for coal measures rock (5 MPa), results in the required crack initiation and re-opening pressures for a range of cover depths as shown in Figure 10.

(1)



Figure 10: Pressures required for crack initiation and crack re-opening in moderate strength (5 MPa Tensile strength) rock for Australian real mine virgin stress conditions (Purcell *et al*, 2016)



Figure 11: Concept of hydraulic fracture growth in rock (Mills and Jeffrey, 2002)

Crack initiation pressures ranging from almost nothing to 50 MPa are predicted and these are used by Purcell *et al* (2016) to conclude that crack initiation pressures in coal measures strata are both highly variable and require site specific consideration in terms of stress conditions and rock parameters before applying roof bolting systems that include resin pressure reduction measures.

The analysis conducted and presented by Purcell *et al* (2016) is fully agreed with in terms of crack initiation pressures required to fracture solid rock material around a vertical borehole in virgin conditions whereby the *in situ* major and minor horizontal stresses act across the borehole plane. However, the analysis significantly over-predicts crack initiation pressures in the immediate roof of a mine roadway on the basis of the following:

- i. Crack initiation develops perpendicular to the minimum applied stress. In the case of the bolted interval above a mine roadway, the minimum stress is inevitably vertical due to the presence of the underlying roadway void, hence the fracture propagation is likely to be horizontal as illustrated in Figure 11 from Mills and Jeffrey (2002).
- ii. The magnitude of the vertical stress within the bolted interval of a mine roadway is inevitably substantially less than the *in situ* vertical stress or either of the principal horizontal stresses

due to the existence of the underlying roadway void which acts as a very efficient vertical stress reliever. Therefore the confining stress (σ_2 in Equation 1) to be overcome during roof bolt installation is likely to be far lower than indicated by the analyses of Purcell *et al* (2016). The same logic was applied by Mills and Jeffrey (2002) in their analyses for hydro-fracturing the spanning conglomerate unit above longwall panels at Moonee Colliery for the purpose of windblast mitigation and control.

iii. The weakest horizontal planes in the roof of a mine roadway are inevitably bedding planes and contacts, which have a tensile strength in the vertical direction substantially less than 5 MPa (as assumed in Figure 10). Frith (2012) reports the tensile strength of bedding planes in both sandstone and carbonaceous material as found from direct tensile testing of core samples. In all cases, average values were less than 0.5 MPa and even for solid sandstone, the tensile strength was found to be just over 2 MPa. Therefore, the assumption in the Purcell *et al* 2016 analysis that a 5 MPa tensile strength represents "moderate" strength rock, is judged to be a significant over-statement and certainly, is an order of magnitude higher than the tensile strength across horizontal bedding planes, this being the more relevant consideration in terms of the stability of a bolted mine roof.

For weak bedding planes/contacts within the bolted interval, typical crack initiation pressures in the order of 3 MPa and less are estimated to be far more realistic, such values being (a) depth independent due to the very low vertical stresses acting within the bolted interval being almost entirely determined by the formation of the underlying roadway void and (b) at the low end of resin pressures that have been measured in surface and *in situ* bolt installation testing, including those reported by Purcell *et al* (2016).

One further aspect in regards to resin pressures driving the development of bedding plane separations in the roof needs to be considered, namely the short time period (of only a few seconds) that high resin pressures are able to act. Griffith Crack Theory states that the highest stress is required to start the propagation of a crack, but once initiated the stress required to further propagate it decreases as a function of the length of the crack. Therefore, in the example of resin pressures causing bedding plane separations in the roof of a roadway, it may be that the main significance of resin pressure is simply to commence the propagation of a fracture that would have not otherwise started under the action of horizontal stress alone, but once started the horizontal stress is then able to drive its further propagation unassisted.

In terms of the controls on resin pressure development, if the problem is considered as a piston being pushed into a closed void space full of resin, resin pressure will develop if the rate of resin volume escaping back past the piston is less than the volumetric compression of the resin ahead of the piston. Therefore it is necessary to consider both the rate of piston insertion (roof bolt insertion rate in this instance) and the various factors that act to restrict the escaping of resin back past the piston (the roof bolt in this case). It is self-evident that slowing down the rate of roof bolt insertion into the bolt hole will tend to reduce the development of resin pressures ahead of the bolt, as this allows more time for resin to escape past the bolt. The data in Figure 6 relates to an insertion rate of 128 mm/second and the test data shown in Figure 12 (developed in conjunction with DSI) was typically associated with a bolt insertion rate of 150 mm/second.



Figure 12: Resin pressure measurements for varying hole diameters and resin volumes

Rate of bolt insertion is also a relevant consideration in terms of resin mixing as if the rate of insertion is too slow (say 100 mm/second), for a 1200 mm or 1400 mm long resin cartridge as commonly used by industry for full encapsulating a 1.8 m or 2.1 m long bolt, the bolt will not reach the back of the bolt hole and be spun sufficiently (according to suppliers specifications) to ensure adequate mixing in the top section of the bolt without over-spinning the resin in the bottom section of the bolt.

Reducing the rate of bolt insertion is an obvious method for lowering resin pressures developed during bolt installation, as implied in the test work reported by Purcell *et al* (2016) who conduct their testing at an insertion rate of 100 mm/second and report lower resin pressures than other published test data including Figure 12. However it is almost certainly inconsistent with resin mixing needs along the full length of the roof bolt. Therefore other remedies are required to reduce resin pressure development, which leads to the various reasons why resin is restricted from flowing along the annulus around the roof bolt during insertion.

Figure 12 contains typical results from a series of roof bolt installations under controlled conditions conducted by the authors and DSI. The first point to make in regards to this type of testing is that it is vital to use a closed system so that resin cannot escape by means other than back past the bolt being inserted, this ensuring that the maximum possible resin pressure is measured. Similar testing conducted *in situ* for example, is prone to resin bleed off through any openings in the roof strata, particularly in friable roof types such as coal/claystone sequences, and will inevitably return lower resin pressures than would be the case in a closed system. Such testing is judged to be meaningless if the roof bolting system design objective is to minimise resin pressures so as to prevent the development of open fractures in the roof in the first instance. This is assessed to be a major oversight of the *in situ* testing reported by Purcell *et al (*2016) and fully explains the very low measured resin pressures which are used in isolation to then discredit the significance of resin pressures and their potential detrimental influence on roof instability.

Figure 12 demonstrates the significance of two key drivers of resin pressure development during bolt installation:

i. resin pressures decreases as a direct function of increasing bolt hole diameter (i.e. annulus thickness around the bolt)

ii. resin pressures decrease as a direct function of using less resin as evidenced by the pressure development curve associated with the use of a 440 mm long resin cartridge as compared to 1000 mm.

Two further logical drivers of resin pressure during roof bolt installation are the viscosity of the resin and the extent to which the rotation of the roof bolt during installation acts to 'pump' the resin back up the hole thereby further restricting its flow past the bolt.

With regard to any pumping action of the spinning roof bolt, it is noted that the early roof bolt patents included a specific innovation whereby the deformed profile was designed to push resin back up the hole during spinning in, as opposed to having it pump resin out of the hole. Therefore the idea that a spinning roof bolt acts to prevent resin flowing along the annulus during installation has been a roof bolt design characteristic since their first use in the mining industry. This pumping action can be readily eliminated by either the use of a smooth bar or preferably, a neutral deformed profile such as a herringbone pattern, which neither pushes resin back up the hole nor pumps it out of the hole. The general relationship between the efficiency of a pump and minimising the clearance between the impellor (in this case the roof bolt) and casing (in this case the bolt hole wall) is also noted in the context of the annulus thickness around the bolt.

A link between increasing resin pressure and increased roof fracturing via resin being forced into the surrounding roof strata, can be reliably inferred from various known changes in bolt encapsulation according to either increasing the bolt hole diameter or volume of resin used, both having previously been inferred to influence resin pressure development.

Figure 13 is taken from Craig (2012) and it is clear that for a 1200 mm resin length, the lower bolt hole diameters result in the encapsulation achieved being less than 100% of the theoretical value, this only being achieved at a hole diameter of 28 mm. Other published research studies mirror this general outcome.

	Resin Length	Drill Bit			
		27 mm Spade	27 mm Angel	26.5 mm twin-wing	28 mm twin-wing
Theoretical	1200	2017	1918	2205	1509
Actual		1480	1420	1370	1530
% resin loss		27%	26%	28%	NIL
Theoretical	1400	2353	2238	2572	1761
Actual		1700 + excess	1700 + excess	1700 + excess	1700 + excess

Figure 13: Resin loss by drill bit diameter (Craig, 2012)

Figure 14 shows variations in what is termed as "Encapsulation Ratio" for varying resin cartridge lengths from 1.8 m long bolt installations in a friable coal roof. For a 28 mm diameter hole, a 21.7 mm diameter bar and a 25 mm diameter resin cartridge, for every 1 mm of resin cartridge length, 1.8 mm of bolt encapsulation should theoretically be achieved if no resin is lost from the hole and the hole diameter is accurate. In other words, if there is no resin loss the Encapsulation Ratio should be 1.8. The data in Figure 14 shows that for resin lengths up to 700 mm, the measured Encapsulation Ratio is just below the theoretical maximum of 1.8, the likely reason for this being the actual hole diameter being slightly greater than the assumed hole diameter (drill bit diameter) of 28 mm. However, for resin lengths > 700 mm, the Encapsulation Ratio incrementally reduces such that for a resin length of 1200 mm, the encapsulation length achieved (960 mm) is actually less than that achieved with 600 mm of resin (1080 mm). Logically, this effect is being driven by ever-increasing resin pressures during installation due to increasing resin volumes, thereby driving ever greater resin losses into the surrounding strata.



Figure 14: Measured encapsulation ratio as a function of resin cartridge length – friable coal roof

The data and arguments presented in this section of the paper lead to the inevitable conclusion that resin pressures developed during roof bolt installation are sufficiently high in the top section of the bolted interval to both initiate and further propagate roof fracturing and associated resin loss. In bedded roof conditions whereby preventing such fracturing occurring is a primary objective of reinforcing roof bolts, this is a judged to be a real and legitimate stability concern. Industry focus on achieving full encapsulation for long bolts (> 1.8 m long) in the smallest possible hole is logically aggravating this effect, which is largely hidden from view during operations. Fortunately, there are some obvious controls for resin pressure development that can be modified to substantially reduce this potentially deleterious effect.

ROOF BOLT PRE-TENSIONING

Pre-tensioning generates an axial tensile force in the bolt and a compressive force against the roof at the plate, without the need for roof movement or more importantly bedding plane separation, the latter being the principal driver of roof beam breakdown. This is why it is referred to as an "active" force as compared to the "reactive" force generated by load transfer. A tensile axial load due to pre-tensioning will be developed along whatever bolt length is able to be freely stretched at the time of nut tightening.



Figure 15: Roof extensometer data – applied roof bolt pre-tension two to three tonnes

The effectiveness of the applied pre-tension in reinforcing the initial 0.5 m or so of roof and the significance of doing so, is clearly illustrated in Figures 15 and 16, these being sonic probe extensometer data from Teralba Colliery in the mid 1990's when increasing roof bolt pre-tension was first being operationally evaluated in industry. Figure 15 shows data related to low levels of applied pre-tension, the salient points being (a) the presence of delamination throughout the entire bolted interval (2 m for 2.1 m long bolts) and (b) the associated time dependent roof behaviour whereby equilibrium is not easily being achieved and displacement levels would undoubtedly trigger a

Development TARP in today's industry. Comparing Figure 15 to Figure 16 which is related to a significantly higher level of applied pre-tension, it is clear that the initial 0.5 m of roof strata contains no obvious delamination, total roof displacement is substantially reduced and the time-dependent trend is far more stable. When it is also noted that the height of roof fracturing in both instances is identical and that in neither case are the 2.1 m roof bolts anchored securely into more stable overlying strata, the significant stabilising effect of generating beam action in the initial 0.5 m or so of roof by preventing delamination using the action of the applied pre-tension, is self-evident.



Figure 16: Roof extensometer data - applied roof bolt pre-tension eight to ten Tonnes

If it is accepted that the condition of the initial 0.5 m or so of roof strata is a key roof reinforcement consideration, it raises the question as to whether it is best reinforced via pre-tension or load transfer, the latter by definition requiring full encapsulation to be achieved whereas the former can potentially be achieved without the roof bolt necessarily being fully encapsulated.

It is contended that the critical aspect of utilising roof bolt pre-tension for roof reinforcing purposes is that it modifies the "end condition" of the roof strata between roof bolts from "pinned" to "clamped", clamped-end beams being 4 times as stable as pinned-end beams (all other factors being equal). This is potentially highly relevant in friable roof types whereby the dominant mechanism driving roof instability is roof deterioration between bolts (guttering and buckling) which can eventually lead to instability across the full roadway width if not adequately controlled.

The concept of different roof beam end conditions is schematically illustrated in Figure 17 whereby a pinned roof beam via full encapsulation and minimal pre-tension effect is compared to a clamped beam developed using pre-tension and a bolt free length that is equivalent in length to a beam thickness that can assist in stabilising the full width roof span. The different roof displacement profiles shown between bolts (u- curve for pinned and double s-curve for clamped) is entirely dictated by the end condition of the beam as defined by the installed roof bolts.





Utilising the applied pre-tension to best possible effect requires that at least three requirements are met:

- i. That the pre-tension generated from nut tightening is as high as possible.
- ii. That the roof bolt plate is able to (a) accommodate the applied pre-tension levels and (b) preferably at least the yield load of the bolt without itself going into yield.
- iii. That the resin anchor above the intended roof interval of bolt pre-tension is able to (a) allow the pre-tension level to be generated by nut tightening and (b) again allows at least the yield strength of the bolt to be generated as a result of any subsequent roof delamination below the resin anchor.
- iv. Only points (i) and (ii) will be considered in more detail in this paper.

In terms of the level of pre-tension generated due to nut tightening, the key issues are (a) the applied torque and (b) the thread system, the latter determining the efficiency of the torque to pre-load conversion and must also remain stable under the dynamic loading and associated heating during nut tightening.

Current day hydraulic bolting rigs commonly use two-speed motors whereby "high rpm-low torque" is used for drilling and "low rpm-high torque" for nut tightening. This makes best use of the available hydraulic power for these two significantly different functions.



Figure 18: Roof bolt pre-tension level variations as a function of thread pitch

Attempting to maximise both the efficiency of torque to load conversion and thread stability during tightening is actually counter-productive as the former increases but the latter decreases as thread pitch reduces. Roof bolts generally use a 3 mm thread pitch (standard M24 thread) which is about as low as pitch can go without thread stripping being inevitable during nut tightening.

Test work evaluating pre-tension achieved as a function thread pitch (Figure 18) for a hydraulic rig generating in the order of 400 N.M (300 ft.lbs) torque, indicated that the combination of a 3 mm pitch and a 1.25 D nut did not always allow the maximum possible pre-tension level to be achieved, whereas at 5 mm and above, it did. The solution to this, without decreasing the applied torque, is to either (a) increase the nut length so as to reduce thread contact pressures thereby making the thread more stable or (b) increase the pitch to at least 5 mm.

As a general statement, with a suitably designed thread system modern hydraulic roof bolting rigs that stall at around 400 N.m (300 ft.l) should be able to reliably generate 12 to 15 tonnes pre-tension due to nut tightening. This is a significant roof bolting attribute that has yet to be fully exploited by industry.

The strength of the head plate is a roof bolt system component that received little attention following the industry move to full encapsulation, the plate being seen as relatively unimportant part of the bolting system as a direct consequence. However in any roof bolting system that uses pre-tension for

reinforcing purposes but may not always achieve full encapsulation, the head plate is in fact a vital component of the system.



Figure 19: Load test arrangement for testing of steel domed washer plate (British Standards, 2007)

The British Standard on strata reinforcement support system components used in coal mines (British Standards 2007) states that a roof bolt plate "*shall flatten under a load of 50% to 70% of the nominal breaking load of the bar...*" and "*allow pull through of the rockbolt, nut and conical seat assembly under a load of 70% to 95% of the nominal breaking load of the bar*". In other words, the plate should be in yield at an applied load as low as 17 tonnes (50%) for a 34 tonne bar (X grade steel) and allow system failure at an applied load as low as 24 tonnes (70%) for a 34 tonne bar. The underlying intent is presumably to protect the rockbolt from tensile failure by limiting the strength of the plate. The direct consequence of this is that the plate loses its elastic stiffness (system stiffness being the key reinforcement consideration) at quite low levels of applied load, which is less than ideal.

The other major problem is that plate testing, as defined in the same British Standard, is undertaken as per the arrangement shown in Figure 19. This is a highly idealised test set-up using a flat surface against the plate. Whilst this may allow representative comparisons between different plate designs, it inevitably provides optimistic plate strength values as compared to when used in an undulating and uneven roof environment. Therefore the stated plate design criteria listed previously that are based on the test arrangement shown in Figure 19, will in fact result in *in situ* plate performance at even lower levels than those specified.

Current standard roof bolt plates are understood to have an ultimate strength rating (or collapse loading) in the order of 24 tonnes, which is exactly 70% of the ultimate strength of an X grade bar. Whether this is directly linked to the British Standard is not known, however it confirms that there is potential, via a stronger head plate, to generate and utilise higher bolt loads within the immediate roof strata as compared to the current situation.

In contrast, the basic load transfer mechanism is shown in Figure 20, the main point being that for axial bolt load to be generated due to bed separation, stable resin anchorages are required both above and below the bed separation. It is therefore instructive to consider the extent by which this reinforcing mechanism is able to work within the initial 600 mm of roof as this will provide further guidance as to the true imperative of achieving full encapsulation to the head of the bolt.



Figure 20: Basic load transfer reinforcing mechanism

Ignoring any contribution from the roof bolt plate, the preferred requirement of the resin anchorage system is to allow at least the yield strength (24 tonnes for an X grade bolt) and ideally the full axial strength of the bolt to be developed via bedding separation effects. For a 600 mm thick immediate roof beam, if it is assumed (for the sake of illustration) that bedding separation occurs at the mid-point of the beam, the resin anchorage above the separation is the majority of the bolt length, but below the anchor it logically can be no more than 300 mm in length. Therefore the question posed is whether a 300 mm long resin anchor, particularly in weak roof strata, has the ability to develop 24 tonnes, if not 30 tonnes of axial bolt load?

A 300 m long resin anchor is the same as that used for short encapsulation pull testing, the objective of using a short anchorage being to evaluate the resin bond rather than the strength of the bolt. This in itself is indicative that it is unlikely that a 300 mm long resin anchor will reliably allow the yield strength of an X grade roof bolt to be developed. Short-encapsulation pull out test results in weak types roof commonly indicate pull-out strengths in the range of 10 to 12 tonnes depending upon resin type and its associated characteristics.

Therefore, load transfer reinforcement within the immediate 600 mm of roof is unlikely to be able to develop more than about half of the yield strength of an X grade bolt. Further illustrations of this are provided in Figure 21 (Gale, 1991) and Figure 22 (Gale and Matthews, 1993) whereby it is clear that the axial loads being developed incrementally reduce towards the top and bottom of the bolt. More importantly, in Figure 22 the roof displacement profile is also shown (based on sonic probe extensometry) which indicates that even though axial bolt load reduces towards the bottom of the bolt, the roof strata nonetheless contains a significant amount of delamination as low as the as-cut roof line. In other words, whilst the driver of axial bolt load generation via load transfer is present throughout the entire bolted interval, the load magnitude being developed in the initial 1 m or so of roof is clearly being limited by some influence.







Figure 22: Combined axial bolt load v roof displacement profile measurements (Gale and Matthews, 1993)

Other points of note in regards to the use of load transfer for reinforcing the immediate roof strata are:

- (a) By definition, load transfer requires bedding planes to open up in order to develop axial bolt load. However, the opening up of bedding planes is also the main driver for beam breakdown and associated roof instability (as previously justified). In other words, the required mechanism of load transfer is directly contrary to the primary roof reinforcing objective, namely preventing the opening up of bedding planes in the first instance.
- (b) Whilst this has never been researched, the role of the plate in supplementing load transfer in the immediate roof is not clear-cut. The loading mechanism for the plate largely relies on relative movement between the strata and the bolt, whereas load transfer attempts to minimise such relative movement. The second graph in Figure 22 clearly shows the bolt in yield above 1 m into the roof, but zero axial bolt load at the plate, meaning that the plate is presumably providing no direct contribution to overall load transfer.
- (c) The base of the bolt is the most likely location for "sliming" of the hole wall due to drilling through any overlying clay bands along the bolt length. This effect is rarely captured in short encapsulation pull testing, but is known to significantly reduce load transfer strengths from those generated without hole sliming.

With all of these considerations to-hand, it is concluded that whilst load transfer has the proven ability to develop the full axial strength of an X grade bolt in its mid-section, it is significantly limited in the lower section which is where the first potentially stabilising roof beam is located. Given the importance of this beam to overall roof stability, this is a less than optimum reinforcing outcome.



Figure 23: Schematic illustration of step-wise development of roof softening with increasing roof displacement



Figure 24: Field Data - roof softening progression with displacement (Gale et al., 1992)

The recognition that the maximum potential axial loading of a roof bolt via load transfer is typically limited to the middle portion of the bolt length, is also at odds with the known "step-wise" progression of roof movement and associated softening starting at the roof line and incrementally moving up into the roof (see Figure 23, which is a general illustration of the data presented in Gale *et al.*, 1992 – see Figure 24). The logic here is that if the immediate roof can be reinforced as a stabilising beam such that its vertical movement is restricted, it will then act to limit the upwards progression of roof softening. This is also beneficial as the higher into the roof that roof softening progresses, the less stable the roof overall and therefore, the higher the level (length and density) of long tendon roof support required to control the roof.

It is concluded that with a suitably designed nut and appropriately rated roof bolt plate, reinforcement of the immediate roof "beam" is best facilitated by the application of bolt pre-tension so as to prevent bed separations, rather than load transfer which relies upon bed separations opening up. Furthermore, the inclusion of a defined bolt "free-length" to ensure that pre-tension is applied over a requisite roof "beam" thickness, is judged to be beneficial when the potential for increased roof fracturing in the upper section of the bolted interval due to the use of larger resin volumes to achieve full encapsulation, is also considered.

SUMMARY

The paper has attempted to demonstrate in selected technical areas that the Australian coal industry's general belief that current primary roof bolting systems are fully optimised with little scope for further improvement, is significantly in error. Furthermore, substantial improvements in reinforcing

effectiveness can potentially be realised to benefit mining operations if geotechnical engineers and mine operators are prepared to embrace such a possibility.

To put the above statement into a more practical context, it is useful to pose the following questions to industry:

- Why wouldn't roof reinforcement improve if the reliability of resin mixing with varying hole diameter is substantially improved?
- Why wouldn't roof reinforcement improve if gloving of the resin cartridge film is minimised?
- Why wouldn't roof reinforcement improve if resin pressures generated during bolt installation are substantially reduced?
- Why wouldn't roof reinforcement improve if the length of the bolted interval directly influenced by high resin pressures generated during bolt installation is substantially reduced?
- Why wouldn't roof reinforcement improve if roof bolt pre-tension levels are increased?
- Why wouldn't roof reinforcement improve if roof bolt pre-tension is reliably applied across a section of immediate roof strata that is sufficiently thick to be able to substantially and positively influence overall roof stability?
- Why wouldn't roof reinforcement improve if the load-capacity of the head plate is increased so that a greater proportion of the available roof bolt strength is mobilised?
- Why wouldn't roof reinforcement improve if load transfer stiffness is substantially improved via the use of a modified resin system?

Mine Advice in conjunction with DSI have taken the view that there is substantial benefit to be realised if roof bolting systems are improved in each of these technical areas. DSI's PEAK Resin Bolting system using a "partially" rather than "fully" encapsulated bolt, is the first significant industry initiative that has incorporated all of these various research findings in a more "balanced" overall bolt set-up. It has found full commercial use at several mines and has allowed a number of substantial operational improvements to be realised (e.g. Hart, 2014) without any negative strata control implications. However, that is another story for another time.

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MECHANICAL DIRECT SHEAR TESTS OF CABLES – COMBINED STRESS RELATIONSHIPS

Kent McTyer¹ and David William Evans²

ABSTRACT: Cables have remained an integral part of underground mining in Australia since the 1970s and many of their properties are well-researched. However, no standardised test is generally accepted for shear - an important failure mechanism for cables; therefore, this fundamental property is not fully defined. Further, the uncertainty means the relationship between shear load capacity and axial tensile load (pre-tension) is not completely understood. This paper begins to fill the information gap by reporting the results from a new test method. A simple, replicable and valid mechanical direct (90°) shear test method has been developed, that intentionally departs from existing reported methods, by not embedding the cable. The preliminary results show a clear relationship between peak shear load and pre-tension magnitude, by eliminating the numerous variables associated with embedded shear test methods. The mechanical test method can thus be used to determine the minimum shear performance of cables under repeatable conditions, but also augment existing embedded cable shear research, by providing the baseline mechanical properties of the cable.

INTRODUCTION

Cables have been a part of ground control in Australian underground mining since the early 1970s (Hustrulid 2001). Cables comprise a number of wires (or strands) in a helical formation around a central wire or wires. This arrangement provides both high axial capacity and flexibility. The flexibility is important as it allows for the cable to be long continuous lengths of typically 4 to 11 m, and yet still be installed in the sometimes restrictive roadway heights of coal mines. It has been generally accepted since the 1980s that rock bolts and cables have the primary objective of increasing rock mass stiffness with respect to tensile and shear loads (Gerard 1983). This improvement in rock-mass resistance to tensile and shear forces is a function of a number of mechanical influences, including the use of compression (via bolt or cable tensioning) as well as the transfer of load from the rock mass to the cables.

Cable suppliers provide product specification sheets to end users. This information is comprehensive for the mechanically-derived tensile properties of the cables, including the Ultimate Tensile Strength (UTS), yield load and elongation. These tensile properties are used for ground support design. However, suppliers do not pass on cable shear properties. This information gap exists for two reasons. First, industry does not readily accept any standardised shear test, and second, ground support design process. Yet shear properties are important for end users because ground displacement can load cables both in tension and shear.

The two existing methods for generating the combined shear and tensile stresses in cables are single shear plane methods and double shear plane methods. These methods involve embedding the cable in either resin or grout in holes of various annulus. Embedded methods therefore introduce additional test variables over pure mechanical tests. The resulting variability in test results has meant that publicly available test data is highly interpretive when used to compare cables. Further, these test methods are expensive and time consuming resulting in low volumes of test data. It is argued that this lack of comparable shear test data has held back the industry's understanding of how the mechanical properties of cables in shear influences the performance of cables in the field. Field performance of

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cables is an increasingly relevant topic as deeper and more challenging ground conditions become the norm in the Australian coal industry.

To fill this information gap, DSI developed its own mechanical direct shear test method. The method aims to provide end users with benchmark shear properties from a reliable and valid test. It will complement data derived from mechanical tensile test methods. The key point here is that until the mechanical shear properties of a cable are understood, it is difficult to make sense of the results of embedded shear methods, which have introduced additional variables that further affect the results.

The test method deviates from previous test methods because it does not embed the cable. Instead, the method has the cable fed through holes cut into two hardened, tight-fitting steel cylinders. Then, a Universal Test Machine (UTM) shears the cable at the interface between the two cylinders. The UTM allows collection of both load and displacement data. A frame was used to pre-tension the cables to a range of loads. This allows collection of the pre-tension and peak shear-load relationship. The results and relationships between the variables measured were evaluated against existing publicly available cable shear information and discussed for their relevance to the underground coal mining industry.

CURRENT CABLE SHEAR TEST METHODS

Two tests methods that replicate the field performance of cables are commonly used in Australia. The single shear test method has been used by Windsor *et al* (1988), Windsor (1992), and Windsor and Thompson (1993), Fuller and O'Grady (1994), Hagan and Mahony (2006), Rock Mechanics Technology (RMT) (2006) (described in BS 7861-2, 2009), and improved upon by Megabolt Australia (Figure 1 Megabolt 2015). The double shear test is detailed in Aziz *et al* 2003, 2004, 2014, 2015, 2016, and is commonly associated with the University of Wollongong (UOW). Both methods embed the cable in resin or grout, then subject the cable to shear load until the wires either fail, or displacement becomes excessive. Readers are referred to the above for further explanation and information on these methods.



Figure 1: The Megabolt single shear test rig (Megabolt 2015)

These test methods have highlighted several key points on the performance of cables in shear. These include:

The embedded cable single and double shear methods result in the failure of the cables in combination bending and tension (Figure 2). This failure mode is representative of cable shear in coal mine strata. However, it is expected that this failure mode will result in higher shear load and displacement compared with mechanical direct shear.



Figure 2: Cone and cup tensile failure of wires (left), and grout de-bonding and concrete block deformation during a double shear test (middle from Aziz *et al*, 2014), and bending and tensile failure during Megabolt Shear Testing (right from Megabolt, 2015)

The angle and direction of shearing has been found to influence the performance of the cable. Hutchinson and Diederichs 1996 (Figure 3) reported the stiffest response was found by a combination of shear and tension (135°) , then direct shear (90°) . The least stiff response was shear and compression (45°) .



Figure 3: Typical results from direct shear tests of cablebolts (after Windsor and Thompson 1993, Windson 1992 and Windsor *et al* 1988) from Hutchinson and Diederichs 1996

The embedded material properties influence the shear load. Similar 21.8 mm diameter cables were embedded in resin and grout and double-shear tested at the UOW (Aziz *et al* 2014 and 2015). The variation in shear load may be explained by the difference in embedment materials.

The length of resin embedment affects the performance of the cable in shear. RMT 2006 found that the greater the length of embedment, the lower the shear load achieved prior to failure (Table 1). Longer embedment lengths resulted in a reduction in variation of measured shear load (RMT 2006). Megabolt 2015 found an embedment length of 1800 mm was required to stop cable de-bonding of non-bulbed cables from causing high levels of shear displacement.

									Single	Shear
Test	Cable Description	Wire Type	Cable Diameter (mm)	Cable Pretension (kN)	Resin or Grout	Embedment Length (mm)	Hole Diameter (mm)	Confining Material	Maximum Shear Force (kN)	Average Shear Force (kN)
Fuller and O'Grady 1994	Flexibolt 21 wire	Plain	23	0	Resin (Chemfix SCP4)	350	27	Steel pipe	410, 470	440
						250			410, 441, 393	414.6
RMT 2006	Reflex 7 wire	Indented	23	0	Resin (AT)	500	27	Steel pipe	382, 389, 373	381.3
						900			358, 361, 367	362.2
RMT	Megastrand	indented	24	0	Resin (AT)	250	07	Steel pipe	384, 414, 424	407.3
2006	8 wire		24			900	21		350, 314, 341	335.1
Aziz et al	Hilti 10 wiro	Indented	01.0	50	Grout	200	20	Concrete (40 MPa)	316.4*	No
2014		Plain	21.0	50	(FB400)	300	20		358.4*	INA
Aziz et al 2015		Indented			Resin			Concrete	391*	Na
	JSS 19 wire	Plain	21.8	250	("oil- based")	300	28	(40 MPa)	441*	

Table 1: Test variables and results of embedded shear tests on 21.8 to 24 mm diameter plain cables.

*double shear tests: calculated maximum single shear value equals half maximum x 0.3 (Aziz, 2016)

Pre-tension levels affect the stiffness of the cable in shear. Megabolt (2015) found that increasing levels of pretension reduced both the shear load and shear displacement. However, double shear testing by Aziz *et al,* (2015) returned contradictory results for the relationship between pretension and shear load. This contradiction is arguably due to the Megabolt test method being more effective in reducing friction across the shear face than the method used in the UOW tests.

It is accepted that annulus has been shown to influence the load transfer properties of bolts and cables. Hence annulus size must be considered when testing embedded cables in shear, because it affects the inherent tensile loads that are produced during testing. It also has an influence on debonding.

Double shear tests are typically performed using three solid blocks, typically concrete or sandstone. The strength of the block material has an influence on the development of bending and tensile loads, and these variables then influence the shear load. Hagan and Mahony (2006) found maximum shear load resistance decreased with rock-mass strength.

The magnitude of confinement of the embedment material (the test blocks) influences the measured load in pull testing (Hyett *et al*, 1992, Thomas, 2012). Due to the tensile loading present in shear tests, the influence of confinement was factored into recent shear test methods (Megabolt, 2015).

Friction across the shear plane increases the shear load in single and double shear tests. The test rig must be suitably designed (such as the Megabolt single shear test method) to reduce friction both during the shearing process and due to pre-tension.

Finally, a host of factors vary across cable products, including steel grade, geometry, wire treatment (indented vs plain), whether it is bulbed or non-bulbed, the number of wires, and the cable lay. Each of these factors will influence cable shear properties.

To summarise, a large number of variables affect the results of embedded cable shear tests. Noting that while the differences between different cables should be the focus of shear property assessments, it is actually often lost in the mix of other test variables. Therefore, it is argued that a standardised test method is critical. However, the problem of test validity first needs to be solved.

CURRENT CABLE SHEAR TEST RESULTS

Published shear test results for plain strand (non-bulbed) 21.8 to 24 mm cable are limited (Table 1). The results consist of:

- A single shear test by Fuller and O'Grady 1994 on the 21 wire flexibolt;
- BS 7861-2 standard single shear tests by RMT 2006 on 7 wire Osborne Reflex cables and 8 wire Megabolt Megastrand cables;
- University of Wollongong double shear tests of 19 wire Jennmar Superstrand cables (Aziz *et al* 2015) and 19 wire Hilti cables (Aziz *et al* 2014).

Even within the limited testing available, a host of factors significantly influence the results. These include: plain vs indented wire, cable diameter, resin or grout type, embedment length, hole diameter and confining medium. Table 1 indicates:

- i. Indented wire cables returned lower shear load than plain wire cables. This is thought to be due to the reduced cross-sectional area of indented strand cables, but may also be due to higher bond strength of embedded indented strands reducing bending and tensile load development.
- ii. The shorter embedment lengths (250 to 350 mm) returned higher shear load than longer embedment lengths. However, 900 mm embedment tests returned the least variance. This may be due to longer embedment reducing pull-through, bending and tensile load development.
- iii. The tests using grout embedment returned lower shear load than those using resin. Resin in this case may provide less stiffness and hence reduced potential for a direct shear.
- iv. Shear load ranged from 314 to 470 kN, with an average of 385 kN from a total of 21 tests.

Table 1 shows significant variation in the test results. This is not surprising given the differences in test machinery and test parameters. Further, interpretation of these test method variables is made difficult because of the lack of understanding of the mechanical properties of the cables in shear.

MECHANICAL DIRECT SHEAR TEST METHOD

The aim is to provide the mechanical direct shear properties of the cable. However, the aim is not to provide an approximation of in-situ cable performance. The reasons for this are:

- i. In-situ performance of cables is a function of a vast number of parameters that are often unique to each mine site. Hence, any laboratory-based testing designed to approximate insitu conditions is highly specific to a small selection of mine sites or conditions.
- ii. While installation parameters will change from site to site, the cable itself will have identical mechanical properties. So while the specification or performance of grout, resin or rock type may change and influence the *in situ* shear performance, the cable itself will behave according to the same inherent mechanical properties.
- iii. Mechanical direct shear is the worst-case shear property of the cable, just as mechanical tensile tests are the worst-case tensile measure. Previous laboratory testing and field experience indicates the cable failure mode will be a combination of bending and tension. Therefore, in practice the cable failure loads will typically be between the mechanicallyderived shear failure load and the mechanically derived UTS.

In general, the mechanical shear test method needs to have the following features:

- Accurate, replicable and valid direct shear test methodology that produces results with minimal variation.
- Shears the cable at 90° without introducing bending or tensile forces.
- Measures the shear load of the cable without (or minimising) shear plane friction.
- Eliminates the influence of resin or grout embedment on shear load results.
- Can evaluate the influence of various magnitudes of pre-tension (axial tensile load) on the cable peak shear load.
- Is cost effective and can be easily conducted providing increased availability of test data.

The test procedure involves:

• Passing a 21.8 mm 19 wire cable (Hi-Ten) through the 22 mm diameter holes drilled in two hardened 4140-grade steel cylindrical jigs (Figure 4). Two methods were used, single shear plane and two (double) shear plane for comparative purposes.



Figure 4: Test cylinders showing slotted sections used for single shear tests (left) and arranged with cable prior to testing (right)

- The cable is free to move through the cylindrical test jigs when tensioned as the jigs are not connected to the frame used to pre-tension the cable (Figure 5).
- The cable is tested without pre-tension (Figure 4 right) and with pre-tension of 10 tonne and 20 tonne (Figure 5). Pre-tension is applied using commonly available barrel and wedges and hydraulic tensioning device.



Figure 5: Cable pre tensioning frame – note the axially loaded cable does not increase loading on the shear plane surface

The inner cylindrical jig is displaced downwards by the Universal Test Machine (UTM) at a
constant rate. To minimise sliding friction, both the inner and outer cylindrical jigs have very
tight tolerances, and oil is used to provide fluid pressure and lubrication. The cylindrical shape
ensures the inner jig is unable to rotate or tilt. These measures reduce the sliding friction and
bending moments inherent to embedded shear test methods.

• The displacement of the inner jig causes the cable to shear at 90°. The tests are continued until either 23 mm displacement is achieved or complete loss of load is recorded. Data collected is load versus displacement, and photographs of the test samples.

MECHANICAL DIRECT SHEAR TEST RESULTS

The results of shear testing are in two forms: visual observations and quantitative data from the UTM.

Observations of shearing

Photographs were taken of the cables after shearing. The photographs indicate that the cables were sheared at 90 degrees in direct shear (guillotine effect). Typical tensile failure indicators -such as necking or cone and cup features - were not observed. Cables that sheared without pre-tension had both a distinctive flat shear face and a high angle (80-90°) shear for individual wires. Five wires on each side of the shear plane had evidence of compression before shear failure (causing wire flattening). However, the wires on the other side of the shear plane retained their round profile (Figure 6).



Figure 6: Typical high angle direct shear of 21.8 mm cable without pre-tension (left) and with 20 tonne pre-tension (right)

Pre-tensioned cables typically had mid to high angle shear faces (60-90°). Compression of outer wires was observed, as was the rounded profile of the wire on the other side of the shear plane. The outer wires were seen to retract away from the shear face after wire failure; logically caused by relief of axial tension. Loss of the outer wire confinement was believed to have caused the inner wire bending. Observed failure mode was essentially the same for single and double shear tests.

Shear load and pre-tension data

The peak shear results for the single and double shear tests at 0, 10 and 20 tonnes pre-tension from the UTM are shown in Table 2. The variation in the results can be due to measurement error in the test method, or individual differences in the product. Resolving the source of the variance will benefit suppliers and end users, because it will either lead to improvement in the testing method or help distinguish between products based on the quality control (variations or lack of) in the measured properties of those products.

Test Type	Pre-Tension (tonnes)	Individual Test Peak Shear Load (kN)	Average Peak Shear Load (kN)	Standard Deviation
	0	321.05	376	17
	0	330.41	520	4.7
Cingle Cheer	10	306.86	207	0.0
Single Shear	10	306.48	507	0.2
	20	233.13	044	0.2
	20	249.71	241	0.3
	0	298.42	204	57
	0	309.84	504	5.7
Double Shear	10	244.90	250	E Q
	10	255.45	200	5.3
	20	206.32	213	6.2

Table 2: Mechanical single and double direct shear load for 0, 10 and 20 tonne pre-tension

The shear load in Table 2 and Figure 7 was recorded during single shear tests. From Table 2 and Figure 7 the following comments are made:

- Shear stiffness of the cables in all tests prior to yield was essentially the same at 75 kN/mm.
- Peak shear load was highest for the non-tensioned cables with an average of 326 kN, followed by the cables pre-tensioned to 10 tonnes with 307 kN, and then cables pre-tensioned to 20 tonnes with an average of 241 kN.
- The variance in results was greatest for the highest pre-tension value of 20 tonnes, and lowest for the cables with 10 tonnes pre-tension.



Figure 7: Shear load and displacement plot of single shear 21.8 mm diameter cables

The shear load in Table 2 and Figure 8 was determined by halving the double shear test load. This was done to account for the higher load caused by two shear planes. From Table 2 and Figure 8 the following comments are made:

• Shear stiffness of the cables in all tests prior to yield was essentially the same at 67 kN/mm.

- Peak shear load was highest for the non-tensioned cables with an average of 304 kN, followed by the cables pre-tensioned to 10 tonnes with an average of 253 kN, and then cables pre-tensioned to 20 tonnes with an average of 213 kN.
- The variance in results was greatest for the highest pre-tension value of 20 tonnes, and lowest for the cables with 10 tonnes pre-tension. Compared with the single shear tests the variance was more consistent across the different levels of pre-tension.



Figure 8: Shear load and displacement plot of double shear 21.8 mm diameter cables

Figure 9 shows the relationship between average peak shear load and pre-tension for the single and double shear tests. From Figure 9 the following comments are made:

- The single shear tests had a higher peak shear load by approximately 30 kN for the given levels of axial load.
- The single and double shear tests displayed essentially the same linear relationship of decreasing peak shear load for increasing pre-tension. The results indicate a 43 to 47 kN reduction in shear load for every 100 kN of pre-tension applied.



Figure 9: Average peak shear load and pre-tension plot of single and double shear tests

DISCUSSION

Shear stiffness

Shear stiffness was 10% greater for the single shear tests than the double shear tests (75 kN/mm vs 67 kN/mm). It is suggested that the higher stiffness was caused by additional friction between the two test cylinders due to rotation (or tilt) of the inner cylinder. There was no visible evidence of friction between the cylinders after double shear testing. However, some evidence of friction on the inner cylinder was observed after single shear testing.

Stiffness was the same for cables that have no pre-tension and for those with 10 and 20 tonne pretension. This confirms that shear stiffness is not affected by cable tension. The non-embedded stiffness results are not directly comparable with previous single and double shear embedded results because those methods contained bending and tensile loading of the cable and shear plane friction.

Shear load and tensile load

The results showed that peak shear load was highest for cables that had no pre-tension applied. Load then decreased with increasing levels of pre-tension. This is thought to be due to a combination of:

- The axial load contributing to the early failure of individual wires due to a combination of shear and tension, and
- Peak shear failure occurring when a smaller number of wires failed when the cable was in tension, but a larger number of wires failing simultaneously when the cable was not tensioned. This may be caused by differential compaction effects in the cable void space. Evidence of this can be seen in the post-peak failure differences shown in Figures 7 and 8.

Past research has shown that peak shear load during embedded shear testing averaged 385 kN for a range of pre-tension loads (Table 1). In comparison, the non-embedded direct shear tests returned an average peak shear load of 315 kN (combining all single and double shear results) when no pre-tension was applied. The lower shear load and displacement of the non-embedded direct shear tests is thought to be due to the lack of bending and tensile loading of the cable. Therefore, the non-embedded direct shear load results are considered the worst-case shear failure mode for cables and thus return the lowest shear load.

The 30 kN difference in peak shear load is relatively constant for the mechanical single and double shear plane test methods (Figure 9). This could be due to:

- Higher friction during the single shear tests caused by tilting of the inner cylinder, and/or
- High localised cable stresses caused by closely spaced shear planes in the double shear test.

Further work is being undertaken to understand the difference in peak shear load.

CONCLUSION

The mechanical direct shear test method is not designed to replace existing embedded single and double shear test methods. These methods remain valid because they offer a simulation of cable shear performance in mines or tunnels. Rather, the direct shear test method adds to embedded test methods in the following ways:

- It is simple and rapid, and repeatable and cost effective.
- By using two tight-fitting cylinders, the amount of friction generated on the shear plane is minimal and a 90° direct shear is achieved.

- Isolating the cable tensioning frame from the shear jig results in no additional friction being placed on the shear surfaces. This provides a clear relationship between different magnitudes of tensile load and shear load.
- It provides the mechanical properties of cables in shear isolated from other test variables.

The shear test method presented in this paper provides the worst-case performance of cables when subject to 90° direct shear in a non-embedded state. The test produces consistent minimum shear values that can be evaluated in the same light as the mechanical tensile tests. With this information end users can:

- Undertake robust comparative assessment of different cable types,
- Undertake embedded shear testing with a greater understanding of the mechanical shear properties of the cable, and
- Use the minimum shear properties either when designing ground support requirements for future underground excavations, or during back-analysis of ground support performance.

Finally, it is noted that this test method is considered a work in progress. Development of the method is required to further understand test variability, maximise its applicability to a range of cable products, and to develop a mechanical direct shear test method that can be used to standardise the reporting of cable shear properties by suppliers.

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QUANTIFYING THE CREEP BEHAVIOUR OF POLYESTER RESIN AND GROUT

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ABSTRACT: Creep of in-situ rock bolts has been apparent and problematic since their introduction in the mining environment. If enough creep is experienced, the bolt has the possibility of failure leaving the opportunity for roof instability. In an attempt to quantifying the creep behaviour in polyester resin and cement grout, laboratory-testing procedures were developed. It wasdecided that two separate tests would provide the data needed to fulfil the scope and objectives. The tests chosen were UCS machine deformation testing and Laboratory Short Encapsulation Pull Test (LSEPT). Based on past research in the scope of the project, a methodology was developed along with measuring techniques to accurately monitor the deformation. Based on the data analysis, the displacement for each sample from the pull test suggested that water based resin deforms the most under an induced load whereas grout tends to deform the least. The long term creep test yielded a peak strain of 0.72% and 1.11% for oil and water based resin respectively. Further calculations concluded that oil based resin had the highest resistance to failure with a shear strength of 8.47 MPa, whereas water based resin yielded a shear strength of 5.5 MPa.

INTRODUCTION

Rock bolts and cables bolts have become one of the main forms of support for most geotechnical excavations in the modern age. They are used due to the high level of successful implementation over the past few decades. Rock bolts consists of a steel rod inserted into a drill hole with an anchor at one end and faceplate and nut at the other. Once in the hole, the void around the bolt is filled with a bonding material, which bonds the bolt to the surrounding rock mass. The bolt is then tensioned to a specific load to support the excavation. Cable bolts are installed in the exact same manner, however the cable consists most commonly of 7 wire strands woven together to form a strong steel cable (Hem, 2014).

Various studies and tests have been done to evaluate and test the performance of bolts in the underground environment. Limited tests however, have been conducted on the specific subject of bolt creep. Creep is defined as a measure of deformation due to an induced stress which is less than the yield stress (Mirza, 2016). The key area associated with creep within the bolting process is the bonding interface between the bolt and the surrounding rock mass. Since the bolts are pretensioned to a specific load, they can experience deformation sometime after the installation. The creep in roof bolts generally leads to the loosening of bolt caps which can result in localized instability of the roof. Bolts can be anchored in a couple of different ways.

Mechanically anchored bolts comes in two forms, slot and wedge bolts or expansion shell bolts, both of these anchor themselves in the strata by the means of expanding when installed. These bolts however, had many problems that led to lost efficiency from anchoring in weak sedimentary layers and a considerable amount of creep was experienced especially when blasting took place. In an attempt to overcome some of these problems, chemically anchored bolts were introduced. The main objective of these bolts was to improve overall bolt performance. The performance was greatly improved as early tests demonstrated an increase in support stiffness (Mieczyslaw, 2000). The stiffness of the chemically anchored supports was improved due to the increased anchorage length and bond strength between the bolt and rock interface. Polyester resin is mostly used as the chemical anchor for rock bolts. Resin relies on its shear strength to resist bolt movement within the bore hole. It

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is important for the bolt-resin and the resin-rock interface to bond properly as this would affect the stiffness of the support (Hem, 2014).

SIMILAR TRIALS

As part of a time-dependent deformation project, the University of Wollongong (UOW) undertook testing using two commonly used cement grout products. The project was aimed at testing the Uniaxial Compressive Strength (UCS), elastic modulus and creep of each sample. All samples were tested in the lab using a standard test developed by the UOW. Two samples were loaded to 100 kN in compression for 15 min. It was found that the samples did not experience any significant creep in the short term with the highest recorded strain value being only 0.27%. The difference in creep experienced between the two samples was found to be only 0.04%, which is a very insignificant value. The main limitation of this test, is that the creep was measured over a very short period of time, 15 min. Therefore, the standard test procedure in this paper would be used in the research project but would be adjusted to test the long-term creep effects of various resins and grout (Mirza, 2016).

The International Journal of Rock Mechanics and Mining Sciences posted an article which discussed the performance testing of fully encapsulated resin bolts. A test was developed to quantify performance of these bolts based on some field-testing and mostly laboratory load-displacement results. The bolts were installed in an underground environment and were overcored to retrieve samples ready for laboratory testing. Testing of overcored samples gives a good indication of the geological characteristic within the study area and it provides an indication of stiffness, peak load and residual loads. Overcoring also provides other important information regarding the bolt system such as resin mixing effectiveness, problems with gloving, resin migration and over drilling of holes. Some very fractured samples were recovered from the study area however, short samples of 300 mm, in sections where the rock is less fractured, were prepared for laboratory testing. The paper concluded that the weakest region in the bolt encapsulation is the toe area. Results proved that bolts experienced more deformation at this location due to poor mixing and it suggested that the performance of a bolt is highly dependent on the resin mix quality and bolt plating. By knowing this, special attention should be given to these areas when preparing samples for testing in the proposed project. The journal overall has relevant information which supports aspects of the project however, tests were done using the same resin. This means that different types of resins could not be compared which is one of the objectives in the proposed project (Villaescusa, Varden, and Hassell, 2007).

Both these trials on grout and resin samples were found relevant to the project since they address key areas to which special consideration should be given. The experimental investigation on grout proved that it should behave brittle and subsequently experience small amounts of creep. It also provided insight to an experimental procedure for creep testing. The trail on resin performance concluded that the strength is highly depended on the mix quality and thus by knowing this, poor mixing practice can be minimised to ensure maximum strength when testing is conducted.

METHODOLOGY

The University of Wollongong published a paper on creep in 2016 in which some industry standard test procedures were used to test creep effects in grouts. Wollongong established a method in which 50 mm cube samples are loaded in a UCS machine at a 100 kN load over a time period of 15 min. The 100 kN or 62.5 MPa load was induced within the first minute of loading meaning the loading rate is very fast. The disadvantage of this procedure is that it tests creep over a very short-term period. Thus, in order to test creep over a longer time period, the test method used as part of this project was based on the test from Wollongong but adjusted to a time period of one month. The test procedure would be repeated three times for two different resin samples. Based on a review of deformation testing, a second procedure was developed as part of this project to test the total amount of displacement experienced by an encapsulated bolt. Past studies by Aziz *et al* (2014) concluded

that strength increases with curing time however, the second procedure aims to quantify the amount of creep experienced if load is applied early after bolt installation. The Laboratory Short Encapsulation Pull Test (LSEPT), was chosen to evaluate creep using a pull out load which is based on an industry pretension of 8 t. Samples would be encapsulated such that the bond length is 100 mm. This test would be carried out over a period of one week with regular measurement intervals.

Deformation testing (UCS machine)

The deformation testing was conducted using a UCS machine with strain gauges attached to each sample. Test samples are 50 mm cube in size. The testing procedure is as follows and Figure 1 shows the test schematic:

- 1. Prepare three samples of the same type (resin or grout) in a 50 mm cube mould and make sure they are the same dimensions.
- 2. Attach two strain gauges to each sample that feeds back to a data logger.
- 3. Set up the samples in the UCS machine one above the other. Ensure that the samples are directly aligned vertically.
- 4. Set up the data logger to start recording data.
- 5. Set the UCS machine to a 75 kN load with a loading cycle of 10 kN/min.
- 6. Leave samples loaded for up to a month to record long-term creep effects.
- 7. Repeat the above steps once for each material type.



Figure 1: UCS test schematic

Rock bolt pull out test

The rock bolt pull out test was carried out over a period of one week and the interest in this test is to quantify creep of the bond materials relative to each other. The test procedure is as follows and Figure 2 shows the test schematic:

- 1. Prepare a 100 mm long threaded steel cylinder of 27 mm diameter and a rock bolt, 440 mm in length, for each test material.
- 2. Mix the bonding material according to industry guidelines and standards.
- 3. Centre the rock bolt in the cylinder and pour mixture for the full length of cylinder. Wait until resin/grout is fully set before proceeding to load the sample.
- 4. Set sample in load rig. Apply a constant load of 8 t (80 kN).
- 5. Measure drop in pressure and deformation daily for a week at a time.
- 6. Record data in a table for each of the samples.
- 7. Repeat this test three times for each material type to acquire consistent data.



Figure 2: Pull test schematic

EXPERIMENTAL RESULTS

Long term creep

The recorded data from the data logger and logbook were combined in the excel spreadsheet to produce a graph showing the deformation over the 28 days of the test. During the test it was noted that four of the strain gauges exceeded the maximum designed strain however, enough data for each sample was recorded to produce valid results. Figure 3 below displays the strain of the oil and water based samples over the duration of the test.



Figure 3: Long term creep

It can be seen from the graph that both the water and oil based samples experienced similar strain trends. After the initial loading of the samples, the strain increased to a peak value which thereafter, it reduced to a residual strain. The peak strain obtained for the water and oil based was 1.11% and 0.72% respectively. The nature of the graph shows that quite a significant amount of strain is experienced within the initial stages of loading on the sample. The strain experienced during first loading cycle is called initial elastic strain.

The second phase in long term creep is known as primary creep. This can be seen in Figure 3 from day one till about day six. The rate of creep is high during the early stages of this phase but decreases with time. The third phase, called steady-state creep, is where the rate of deformation follows a near linear increasing trend. This can be seen from roughly day six till day nine for oil based

resin and from day five till day 17 for water based resin. During this stage in the test a peak strain was reached. The final stage of creep was not showcased in Figure 3 and is known as tertiary creep. During this stage the rate of creep tends to increase rapidly as microstructural damage had sufficient time to propagate and generally culminates in failure. It is believed that the microstructural damage develops during the steady-state phase and once the rate of creep increases due to the interaction between the micro fractures, the material enters the tertiary creep stage (French, 1991). During the test done as part of this project, the resin did not experience a tertiary stage which follows the standard trend. From Figure 3 it is evident that during the final stage the strain was steady for a few days which thereafter, the rate decreased to a final residual value. This can be seen as an error encountered during the test, the load was reapplied to 75 kN before every measurement was taken. This partial unloading and loading influences the total strain experienced by the sample. Figure 4 gives a graphical representation of the concept behind loading and unloading of a sample.



Figure 4: Strain effects of loading and unloading on a material (Tanaka, 2003)

As seen in Figure 4, the steady unloading of a sample results in stress relaxation over time which leads to a reduction in strain. This reduction is known as strain restoration. Repeating the stress relaxation for a number of cycles would yield a peak and residual creep strain. The results from the long term creep test show exactly this, confirming that the effects of stress relaxation was the reason for the experimental creep graph only partly representing the trend from a standard creep graph. The residual strain was found to be 0.97% and 0.60% for water and oil based resin respectively.

Pull out test

The results gathered from the pull test was analysed to show the difference in displacement, shear strength, peak load and strain experienced by each of the three materials. A total of three tests were completed on each of the materials. The total displacement for each sample was recorded during the seven days of testing. After each test, the samples were loaded until failure which was dictated by a rapid increase in deformation with no increase in load. To conclude the validity of the test the failure interface was identified. For the pull test to be valid, failure needs to occur between the material and bolt interface. Each of the test samples failed in this manner, thus ensuring the validity of the tests. None of the oil based resin samples failed under the normal testing conditions. Each sample deformed for a full seven days without failure. Figure 5 displays the deformation results obtained for the three oil based resin samples.



Figure 5: Pull test results for oil based resin

The results for each of the oil based samples were found to be fairly consistent. All three samples followed very similar deformation trends over the duration of the test and experienced a maximum displacement between 1.5-2.5 mm. The samples for water based resin showcased a higher displacement when compared to the oil based. The trend for the samples was fairly consistent and nearly linear. Figure 6 displays the displacement results for each of the water based samples.



Figure 6: Pull test results for water based resin.

As seen in Figure 6, the results from the water based samples were linearly increasing. Samples two and three showed very similar results with both trends being near linear. These two samples also experienced the highest displacement of all the tested samples from the pull out experiment. Initially, three water based samples from the pull test failed prematurely. After this, three additional samples were casted for testing and their results were used in the analysis. The premature failure in the waterbased samples could have been caused by the presence of air bubbles in the resin mixture. The water based resin blend was found to be pastier than the oil based mixture which in turn makes it harder for air to escape the resin. Two of the three samples tested for grout did not fail under the test conditions. The grout was found to behave more brittle than the resin samples. Figure 7 graphical shows the results.



Figure 7: Pull test results for grout.

Figure 7 shows the deformation experienced by each of the grout samples over the test duration of seven days. It can be seen that the rate of deformation is fairly linear for most of the samples. The third sample, which failed prematurely during the second day of the test, showcased the same trend as observed in the early stages of the second sample. The relative slow increase in deformation for the samples shows that the grout is quite brittle when compared to the resin samples.



Figure 8: Average creep trend for each material.

Figure 8 displays the average creep trend for each of the materials. From the graph it can be seen that the oil based resin and grout samples yielded very similar results and showcased a low creep rate. The water based samples had the highest rate of creep and produced a near linear increasing trend. These samples also experienced the highest displacement at failure. In contrast, the grout samples yielded the lowest creep rate and consequently had the lowest displacement at failure of the three materials.

Shear strength and strain

To further compare the test samples to one another, the shear strength and peak strain was calculated for each of the materials. To ensure accuracy, the peak strain was calculated by subtracting the elastic elongation of the bolt itself under the peak load. This ensured that the true displacement in the test material was used for the strain calculation. The shear strength was calculated based on the embedment length, bolt embedded surface area and the peak load experienced by the sample. It is know that failure from a pull test is largely caused by shear, but has some component of torsional unscrewing (Cao, 2012). Due to the complexity of analysing torsional unscrewing of the bolt, failure was assumed to be caused by shear only. Figure 9 below displays the averaged shear strength calculated for each of the materials.



Figure 9: Shear strength of bonding materials

It is clear from the results that the oil based resin yielded the highest shear strength out of the tested materials. It experienced peak failure loads of 135 kN on average which is much higher than the 72 kN and 90 kN experienced by the water based resin and grout respectively. The grout samples yielded an average shear strength of 5.5 MPa, which was slightly higher than the water based resin but lower than the oil based resin. Subsequently the water based resin yielded the lowest shear strength of 4.5 MPa. Figure 10 displays the associated peak strain experienced by each of the materials.



Figure 10: Shear strain of bonding materials

The strain experienced by the samples is representative of the overall material stiffness. The rate of creep was found to be highest for the water-based resin and lowest for the grout. The calculated peak strain for each material suggests that the grout is most brittle; the oil-based resin is more brittle than the water based but more ductile than the grout and the water based resin is most ductile. Therefore meaning that under an induced axial load, water based resin would experience the highest amount of deformation out of the three materials. In summary, the results found from the pull test suggest that the oil-based resin has the highest shear strength and subsequently has the highest resistance to failure. The water-based resin was found to be the weakest in terms of shear strength and behaved most ductile out of the three materials and the grout samples yielded the lowest creep rate and lowest peak strain.

Influencing factors

Factors that had the possibility of influencing the outcomes of experiments were identified. These factors were considered to be a combination of or be related to mixing practises, mixing ratios, ideal conditions and equipment limitations. Research concluded that bonding material strength is greatly affected by the quality of mixting. All the samples for this project were mixed by hand. Each batch of material was mixed for at least 2 min before pouring it into the mould. In industry, the bonding material is mixed with high speed mechanical mixers to ensure that the components are properly blended together. Since mixing was conducted by hand during the sample preparation stage, some components might not have been thoroughly mixed resulting in a decreased material strength. This could have lead to some form of inaccuracy in the test results.

It is also important to note that the ratios used for mixing play a vital role in the performance of the materials, especially *in situ*. For example, grout is mixed beforehand with a large mechanical mixer and is then pumped into the borehole. This means that the mixture quality should be good which enhances material performance. The resin on the other hand is mixed inside the borehole for only a few seconds, which means the mixtures can vary quite significantly in terms of quality. This varying

quality in the mixing of resin is very hard to get consistent due to the amount of factors influencing the physical mixing. These include but are not limited to borehole diameter, borehole roughness, glove fingering and drill operator, to only name a few. The perfect ratio of mastic to catalyst for a consistent mixture would be 50:50. This means there is the same amount of mastic as there is catalyst. However, the resins used in this project had a mix ratio of 93:7 for oil based and 80:20 for water based. From these mixing ratios, one would expect that given the same environmental conditions, the water based resin should have a better mix quality due to the ratio being closer to 50:50 when compared to oil based resin. This means that although the experimental results show that oil based resin, when perfectly mixed, might be stronger than water based, this might not be the case in-situ since the mixing quality has a higher chance of being good in the water based resin than in the oil based.

Another important factor to consider is that the results obtained from the experimental investigation was conducted in what is called an ideal environment. Most of the factors in an ideal environment can be controlled. This means that influencing factors such as improper mixing, glove finger and borehole inconsistencies had been minimised to obtain peak results for each of the materials. Further *in situ* testing of the resins and grout could provide varying results due to the introduction of the other external influences.

Limitations in the available equipment for the experimental testing procedure had a big influence on the number of samples tested and also the accuracy of the results that were obtained. For the long term creep test, carried out in the UCS machine, only three samples of each type could be tested at a particular time. With this test having a duration of 4 weeks, it makes getting consistent results through repetition very time consuming. The availability of one compression test machine limited the amount of tests that could be carried out during the timeframe of the project. Another aspect of the UCS machine that influenced the results is the fact that the load of 75 kN could not be sustained for the duration of the test.

The load had to be reapplied before every measurement was taken. This introduced stress relaxation which led to the test not following a standard creep curve. For the pull test, the equipment was found to be performing well overall. Hhowever, the load on the bolts was reapplied to the required load of 80 kN between measurements and since only one hydraulic ram was available for conducting the pull test, only three samples of each material could be tested in the project timeframe. In terms of measurements, load readings were conducted based on the hydraulic gauge attached to the ram. A load cell for more accurate measurement of the applied load was only acquired during the final stages of testing.

CONCLUSION AND RECOMMENDATIONS

Time-dependent deformation as a result of an induced load which is less than the yield strength of a material can be referred to as creep. The creep can be presented on a graph showing the amount of strain experienced over time. Resin bolt performance testing showed that the weakest region in the bolt encapsulation is the toe area. This is directly related to poor mixing practices thus, special attention should be given to mixing for sample preparation.

Long-term creep testing on water and oil based resin yielded a peak strain of 1.11% and 0.72% respectively. The trend of creep in this case did not conform to a standard creep curve since stress relaxation was introduced due to loading and unloading of the samples over the duration of the test. From the rock bolt pull test it was concluded that grout behaved as most brittle while the water based resin behaved as most ductile. The peak strain experienced by the samples were 2.5%, 3.2% and 1.9% for the oil based, water based and grout materials respectively. The highest shear strength was recorded for the oil based resin at 8.47 MPa. This compared to the 5.5 MPa for grout and 4.51 MPa for water based clearly showing that the oil based resin resisted the highest load. The peak average load at failure for each of the samples was 135 kN, 72 kN and 90 kN for the oil based, water

based and grout respectively. The bonding material found to experience the highest amount of creep was the water based resin and the lowest amount was recorded for the grout.

Based on the results obtained, it is recommended that:

- Further tests to be undertaken on each of the bonding materials to increase the consistency of the current results and especially for the water based resin. Obtaining multiple repetitions of the same test would ensure that the results gathered are accurate for the specific material.
- Improved equipment to be acquired that overcomes the mentioned limitations. Especially the limitation of not being able to sustain a constant load.
- Bonding materials to be tested *in situ*. This will introduces a range of external factors not
 easily controlled and may produce different results. These results can then be used to be
 directly advised to industry.
- When preparing cube samples, the moulds should be vibrated to release any trapped air bubbles as this would reduce the amount of strain error encountered during the test.
- Mechanical mixers to be used during sample preparation to ensure each of the resin and grout batches are blended thoroughly.
- The effect of industry installation techniques on the shear strength of the material should be analysed through borehole testing.

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A COMPARISON BETWEEN RESIN AND A CEMENTITIOUS MATERIAL IN THE GROUTING OF CABLE BOLTS

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ABSTRACT: The mining and civil underground construction industries have increasingly become reliant over recent decades on the use of grouted cable bolts for ground support especially in difficult ground conditions. Despite this there are still some key areas of cable bolt performance that are poorly understood. This paper details an investigation that compared the anchorage performance of a plain strand cable bolt that was grouted in a confining medium using in one case a resin grout and in a second case a cementitious grout. The impact of borehole diameter on the performance of the two grout types was also studied. The investigation involved a series of twenty pull-out tests with a plain strand bolt using the UNSW modified Laboratory Short Encapsulation Pull-out Test. In tests with the standard borehole diameter of 27 mm, it was found that the resin grout exhibited a lower average peak load carrying capacity than the cementitious grout. By contrast, with the larger diameter of 37 mm, the resin grout outperformed the cementitious grout in terms of average peak load carrying capacity. With a cementitious grout, an increase in borehole diameter size from 27 mm to 37 mm decreased the average peak load while the load doubled when using the resin.

INTRODUCTION

Cable bolts have been used extensively in the civil and mining industry since the 1960's to support ground excavations. A geotechnical engineer must select from cable bolts of differing length, shape, diameter, strand-configuration, strand surfaces and structural modifications including bulbs, bird-cages, etc. The performance of a cable bolt can alter significantly with these design variations. An engineer must also take into account factors including borehole diameter, embedment length and the type and strength of grout used, etc. which also contribute to the performance of the cable bolt.

The function of cable bolts has been defined as active excavation reinforcement through the use and conservation of the "inherent strength of the rock mass surrounding the excavation" (Villaescusa, Windsor and Thompson, 1999).

Failures of a cable bolt in axial loading can be divided into four categories as illustrated in Figure 1. The first type of failure is failure of the cable-grout interface. The mechanical interlock between the grout and bolt depends largely upon the surface roughness of the bolt. This type of failure is the most common type of failure observed in-situ and during pull out tests (Rajaie, 1990; Hutchinson and Diederichs, 1996; Hyett *et al*, 1995; Hyett, Moosavi and Bawden, 1996; and, Singh *et al.*, 2001). The second type of failure is the material failure of the grout. This is governed by the strength of the grout. The third type of failure is caused by the failure of the mechanical interlock between the grout and face of the borehole wall. The fourth type of failure occurs when the rock mass immediate to the cable bolt is unable to support the load developed in the cable bolt.

As reported in Hagan, Chen and Saydam (2014), a variation of testing methodologies have since been developed. The latest axial testing methodology is referred to as the UNSW laboratory short encapsulation pull-test (LSEPT) developed by Hagan *et al* (2015) and overcomes a number of disadvantages of previous testing methodologies. Li (2016) has additionally incorporated shrink wrapping to approximately the bottom 90 mm of the cable bolts to ensure constant embedment length as the cable bolt is pulled out during testing.

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Hoek, Kaiser and Bawden (2000) reported on load-deformation results obtained by Li and Stillborg (1999) on steel rebar rockbolts comparing cement grout with resin grout. Li and Stillborg (1999) found both cement grouted and resin grouted steel rebar to have same maximum load and deformation at failure – 15 t and 1.5 mm respectively. The results are shown in Figure 2. This study offers early insights into possible failure styles of cable bolts in resin versus cementitious grout.

This also aims to identify and analyse the effect of borehole diameter on the performance of a cable bolt when using different types of grout. Thomas (2012) as well as Hagan *et al.* (2015) studied the impact of changes in borehole diameter and both concluded that an increase in borehole diameter increased the pull-out load capacity of modified cable bolts. Conversely, an increase in borehole diameter decreased maximum load carrying capacity of plain strand cable bolts in both studies. Thomas (2012) and Hagan *et al.* (2015) used cementitious grout.



Figure 1: Four modes of cable bolt failure (Thomas, 2012)



Figure 2: Load-deformation results by Li and Stillborg (1999) comparing resin vs. cement grouted steel rebar (Hoek, Kaiser and Bawden, 2000)

In contrast, Mosse-Robinson and Sharrock (2010) and Rajaie (1990) also studied the impact of borehole diameter in cable bolt performance. Their results found little impact on the peak load

capacity of a cable due to borehole diameter. Hutchinson and Diederichs (1996) modelling also concludes that borehole diameter should have a negligible effect on cable bolt performance.

METHODOLOGY

To compare the performance of a plain strand cablebolt anchored with a cementitious grout and a resin grout in two size boreholes using the UNSW modified LSEPT after Hagan *et al.* (2015) was used. The addition by Li (2016) of utilising heat shrink wrap along the bottom of the cable bolt to maintain constant embedment length was also incorporated into the tests.

Sample preparation

A total of 20 samples were prepared for testing. The first task was to prepare the moulds for casting the confining medium in which the cablebolt is embedded. The effect of sample size was studied by Rajaie (1990) and more recently studied by Ur-Rahman (2014) and Zhai (2015). Rajaie (1990) reported that sample size diameter does not have an effect on maximum pull-out load beyond 200-250 mm while Ur-Rahman (2014) concluded the inflection point was 300-350 mm. Zhai (2015) found the effect of sample size to be negligible beyond 200 mm and 300 mm for Superstrand plain cable bolts and nutcaged MW9 cable bolts respectively. A 300 mm sample diameter was chosen after taking into consideration the above studies. To construct the exterior formwork for the mould a product called 'Spiral Tube' from Ezytube™ was used.

PVC pipes of length 500 mm with the same exterior diameter as the borehole sizes required, 27 mm and 37 mm, were prepared. These were then wrapped with 5 mm silicone tubing as shown in Figure 3. The silicone tubing would create the desired rifling effect on the borehole walls as seen in the field. The rifling effect is essential in achieving an adequate interlock between the grout and borehole walls. Figure 4 shows the PVC pipes and the moulds being affixed to the fibreboard using a high strength silicone adhesive.



Figure 3: Silicone tubing wrapped around PVC pipe to create a consistent rifling effect in borehole



Figure 4: Completed moulds ready for casting

The next step in the preparation process was to cast the confining medium. The cement-based material that was to be used was special ordered with a strength of 10 MPa. Figure 5 shows the cement-based material being pumped into the prepared moulds and the moulds immediately after casting.



Figure 5: Cement-based material being pumped into moulds (left) and, samples immediately after casting (right)

Following this the cable bolts were prepared for grouting by applying the PVC heat shrink wrap to the bottom 90 mm of the cable bolt. The next step was to grout the cable bolts into the samples. The 10 cement grout samples were prepared using Minova Stratabinder HS at water to cement ratio of 0.45. The water to cement ratio was chosen to ensure the UCS of cement grout is the same as the UCS of resin grout as specified by the manufacturer of the resin grout. The resin grout used was 'XXSlow Resin Premix' by J-lok Resins Jennmar. It was prepared as per recommendations by the manufacturer. The catalyst to grout ratio was 93% XXSlow Resin Premix to 7% supplied oil catalyst. The prepared grout was then poured into the boreholes.

The next step was to affix the anchor tube. The steel anchor tube is a rigid terminating device that transfers the applied tensile load from the hydraulic ram to the cable bolt being tested. The anchor tube also forms part of the device that constrains the cable bolt against rotation during the test. The latter is achieved with a 4 mm deep and 70 mm long key slot, cut into the lower section of the anchor tube, as shown in Figure 6. The key of 8 mm thickness, also shown in Figure 6, is inserted between the anchor tube and the bearing plate and prevents rotation of the cable bolt during testing.



Figure 6: Machined anti-rotation key slot in the anchor tube (top) and, locking key (bottom)

To affix the 610 mm long anchor tubes to the cable bolt the anchor tube was filled with grout. The grout used was Minova Stratabinder HS with a water to cement ratio of 0.4. After grouting the anchor tubes the samples were cured for a minimum of 30 days.

Test setup

To confirm the strength of the confining medium, two types of UCS tests were conducted. Firstly, while the samples were being cast, 50 mm cubic grout samples were prepared and cured in identical fashion to the test samples and tested at the time of the pull-out tests. Additionally, after pull-out testing was completed the 50 mm core samples were drilled in the confining medium and tested.

The first step in setting up the test facility was to place the assembled cablebolt and anchor tube within a matched pair of half steel tubes. The tube is meant to provide confinement, reacting against any radial stresses generated in the confining medium as a result of the axial load applied to the cablebolt. The split tubes were placed around the in the confining medium with nuts and bolts used to secure the two halves hand tightened. The 10 mm annulus between the sample and split tubes was filled with grout made using 'General Purpose' (GP) cement at water to cement ratio of 0.5:1. The grout was left to cure for one day before testing.

On the following day and prior to a test, the securing bolts for the split-tube were tightened to a constant torque of $50 \text{ N} \cdot \text{m}$. Figure 7 shows a test sample placed in the confinement split-tube have been placed and the bolts being tightened to the required level of torque.



Figure 7: Confinement split-tubes grouted around sample (left); and, confinement torque being applied (right).

The complete assembly was then placed in the testing facility consisting of a 100mm thick bearing plate to distribute the reactive load evenly across the surface of the confining medium; a hollow hydraulic cylinder placed over the anchor tube that provided the axial pull-out load; a load cell placed on top of the cylinder to directly measure the applied force of the hydraulic cylinder; and a reaction plate screwed to the top of the anchor tube to transfer the load from the cylinder to the anchor tube. Displacement of the cable bolt was measured using a Micropulse linear position sensor. The setup is illustrated in Figure 8. A hydraulic pump was used to power the hydraulic cylinder ensuring a constant displacement rate with full extraction of 100 mm cablebolt within approximately 15 mins.



Figure 8: Illustration of testing facility (illustration adapted from Hagan, Chen and Saydam (2015))

RESULTS AND ANALYSIS

In total, 20 pull tests were conducted with a plain strand cablebolt; 10 test samples using a cementitious grout and 10 with a resin grout. In each case, half the test samples had a borehole diameter of 27 mm while the remaining half had a borehole diameter of 37 mm. In all cases, the failure mode was at the bolt/grout interface.

Strength tests conducted on the confining medium found the average UCS measured of three diamond cored specimens was 10.9 MPa while that of the three cast 50 mm cubic specimens was 8.4 MPa. This strength level was intended to be representative of a weak rock type such as coal.

COMPARISON OF CEMENTITIOUS AND RESIN GROUTS

27 mm borehole diameter

The objective of this investigation was to identify and analyse the difference in performance of the cable bolt when using two different grouting materials, namely resin and cementitious grout in samples having a 27 mm diameter boreholes, this being the supplier's recommended borehole diameter for the cablebolt. Figure 9 shows the results for the cementitious grout and the resin grout with the best three results highlighted in a darker shade. The red curve in each case represents the generalised shape of the performance curve.

Comparing the graphs it is clear that cementitious grout system achieved a significantly higher ultimate peak load. As Table 1 indicates the average peak load for the cementitious grout was on average 40% greater than the resin grout. Both grout types achieved full load resistance over a comparatively short displacement of around 5 mm though the stiffness of the cementitious grout system was nearly 23% greater than the resin grout.

Also, the shape of residual loading curve is unique to each grout material. The load of the cementitious grout continued to increase with displacement over the range of constant embedment length of 90 mm whereas the load remained largely unchanged over the same range of displacement.

The average residual load after 90 mm displacement with the cementitious grout was 47% larger. Interestingly there was more variability in the performance with the resin grout as indicated by higher values for coefficient of variation, conversely the cementitious grout produced more consistent results.



Figure 9: Pullout performance in a 27mm borehole with the cementitious grout (left) and resin grout (right)

 Table 1: Analysis of the anchorage performance characteristics for the 27mm diameter

 borehole

Statistical	Pe	ak Load (I	<n)< th=""><th>Load</th><th>@ 90 mm</th><th>n (kN)</th><th colspan="4">Initial Stiffness (kN/mm)</th></n)<>	Load	@ 90 mm	n (kN)	Initial Stiffness (kN/mm)			
Parameter	Ceme nt	Resin	diff.	Ceme nt	Resin	diff.	Ceme nt	Resin	diff.	
average	117	70	-40%	114	61	-47%	43	33	-23%	
maximum	125	74	-40%	121	69	-43%	58	38	-34%	
minimum	112	61	-46%	110	53	-52%	22	29	+31%	
Coefficient of Variation	5%	9%	-	4%	11%	-	36%	11%	-	

37 mm borehole diameter

A second set of tests were conducted with a 10 mm larger borehole diameter to examine the sensitivity of diameter on anchorage performance. Figure 10 shows the results for the 37mm cementitious grouted and resin grouted samples.

In this case the performance is reversed with the ultimate peak load for the resin grout sample being much higher. Again, the characteristic shape is unique to each grouting material with the load bearing capacity for the cementitious grout again increasing with displacement up to around 60 mm.



Figure 10: Pullout performance in a 37 mm borehole with the cementitious grout (left) and resin grout (right)

Table 2 shows the average ultimate peak load with the larger borehole was 43% higher with the resin grouted sample. Also the residual load at 90mm was 53% higher with the resin grout. It should be noted that the results tended to be less consistent with the resin grout as is evident by the graph in Figure 10 that shows the results for tests were much less than the other three and on par with the results achieved with the cementitious grout. Alternatively, this indicates more consistent results might be achieved when using cementitious grout though this would need to be confirmed with a larger number of test replications.

In terms of stiffness, there was little difference between the two types of grout material, being of the same order as the stiffness measured in the 27 mm with the resin grout.

Statistical	Pea	ak Load (k	(N)	Load	@ 90 mm	(kN)	Initial Stiffness (kN/mm)			
Parameter	Cement	Resin	diff.	Cement	Resin	diff.	Cement	Resin	diff.	
average	97	143	+47%	88.3	137.3	+55%	32	31	-3%	
maximum	104	145	+39%	98.5	140.9	+43%	51	43	-16%	
minimum	90	141	+57%	80.5	132.3	+64%	23	20	-13%	
Coefficient of Variation	6%	1%	-	9%	3%	-	40%	30%	-	

 Table 2: Analysis of the anchorage performance characteristics for the 37mm diameter

 borehole

EFFECT OF BOREHOLE DIAMETER

Cementitious grouted samples

Comparing the effect of borehole diameter on anchorage performance when using a cementitious grout, it can be seen that performance is degraded as shown in Figure 11.

The two graphs show that increasing the borehole diameter reduced both the initial peak load peak and the ultimate peak load. This is accordance with the results obtained by Thomas (2012) and Hagan *et al.* (2015). It can also be seen that the initial load peak is more prominent in the 37 mm results. The shape of the residual loading also changes. The 27 mm samples reached 90% of their maximum peak load after approximately 40 mm of displacement while in the 37 mm samples, the ultimate load was achieved after around 60-70 mm displacement. In-field this would result in a larger roof displacement if a plain strand cablebolt were installed in a larger 37 mm diameter borehole.

Table 3 summarise the effect of changing borehole diameter on cementitious grouted samples. The average ultimate peak load decreased by 17% and the average initial stiffness decreased by 26% with an increase in borehole diameter. The load at displacement of 90 mm was decreased by 22%.





Statistical	Pe	ak Load (ł	kN)	Load	@ 90 mm	n (kN)	Initial Stiffness (kN/mm)				
Parameter	27 mm	37 mm	diff.		27 mm	37 mm	diff.		27 mm	37 mm	diff.
average	117	97	-17%		114	88	-22%		43	32	-26%
maximum	125	104	-17%		121	99	-19%		58	51	-12%
minimum	112	90	-20%		110	81	-26%		22	23	+5%
Coefficient of Variation	5%	6%	-		4%	9%	-		36%	40%	-

Table 3: Analysis of the anchorage performance characteristics with the cementitious grout

Resin grouted samples

Figure 12 shows the performance graphs when using resin grout in samples with borehole diameters of 27 mm and 37 mm. In contrast to the cementitious grouted samples, increasing the borehole size significantly increased the load carrying capacity of the cable bolt. The initial peak load and the ultimate peak load capacity. However unlike the cementitious grout, the change in borehole diameter did not affect the residual loading behaviour of the cablebolt.



Figure 12: Performance with the resin grout in a 27mm borehole (left) and 37 mm borehole (right)

Statistical	Pe	ak Load (I	<n)< th=""><th>Load</th><th>@ 90 mm</th><th>n (kN)</th><th colspan="3">Initial Stiffness (kN/mm)</th></n)<>	Load	@ 90 mm	n (kN)	Initial Stiffness (kN/mm)		
Parameter	27 mm	37 mm	diff.	27 mm	37 mm	diff.	27 mm	37 mm	diff.
average	70	143	+104%	60.8	137.3	+125%	33	31	-6%
maximum	74	145	+96%	69.0	140.9	+104%	38	43	+13%
minimum	61	141	+131%	52.5	132.3	+152%	29	20	-31%
Coefficient of Variation	9%	1%	-	11%	3%	-	11%	30%	-

Table 4: Analysis of the anchorage performance characteristics with the resin grout

Table 4 summarises results for the 27 mm and 37 mm resin grouted samples. Comparing the two borehole diameters it was found that the 37 mm oversized borehole diameter increased the average initial peak load capacity by 104%. Also, the average ultimate load, at displacement of 90 mm, was increased by 125% in the oversized borehole. There was little change in the initial stiffness though of the resin grout system with borehole diameter.

CONCLUSION

The objective of this study was to determine what influence, if any, the type of grout material had on the load bearing characteristics of a plain strand cablebolt and further whether this was influenced by a change in borehole diameter.

In total, the project entailed a multi-factorial study involving four variables, namely a cementitious and a resin grout and, embedment in the recommended standard borehole diameter of 27 mm and oversized borehole of 37 mm. Each arrangement was replicated five times. The confining medium in which the cablebolt was anchored was prepared in one batch from a cement-based material having a measured compressive strength of 10.9 MPa that is equivalent to a low strength rock such as coal.

In the 27 mm borehole diameter samples, the cementitious grouted samples achieved a 40% higher average initial peak pullout load and a 47% higher ultimate peak pullout load after 90 mm displacement than the resin grouted samples. By contrast, the situation was reversed in the larger borehole with the resin grout having a higher initial peak load and ultimate peak load at 90 mm of 47% and 55% respectively.

Overall in terms of load bearing capacity, the optimum performance was by far achieved with the resin grout in the oversized borehole with average initial peak load and ultimate peak load at 90 mm of 143 kN and 137 kN compared to 117 kN and 114 kN with the cementitious grout in the standard borehole. The results appeared to be less consistent when using the resin grout.

In both cases when using the cementitious grout, the pullout load tended to increase gradually with displacement beyond the initial peak load that was achieved in most cases after nearly 5 mm. In all cases with the resin grout, the pullout load remained relatively constant over the constant embedment length of 90 mm.

Considering stiffness, there was little difference between the different combinations of anchorage systems except in the case of the cementitious grout in the standard diameter borehole which was approximately one third greater than the other combinations.

In all cases, failure was observed to occur at the cable/grout interface. This was largely due to the plain strand cable bolt offering minimum amount of friction between the grout and cable bolt compared to what can be achieved when using a modified cablebolt.

The results offer the industry a more accurate decision making opportunities regarding grout use, leading to enhanced safety and productivity. This project will also contribute towards ACARP led effort to standardise the laboratory pull-out test procedure.

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STRENGTH PROPERTIES OF GROUT FOR STRATA REINFORCEMENT

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ABSTRACT: An experimental study was carried out on grout samples prepared from both Stratabinder and BU100 cementitious products. Samples were prepared with various water to grout ratios and tested for uniaxial compressive and shear strength. Triaxial tests were performed on cylindrical samples to determine values for internal friction angle, cohesion and tensile strength. It was found that the water to cement ratio affects the uniaxial compressive and shear strength of grout. The triaxial test indicated that both internal friction angle and cohesion of Stratabinder do not differ significantly from BU100.

INTRODUCTION

Prior to the late 1940's, a large proportion of roof supports in underground mines in Australia consisted solely of timber deployed along roadways. The fragile nature of the timber was the cause of a considerable roof failures and rib collapses prior to the introduction of roof bolting. The early roof bolts consisted predominantly of a mechanical anchor positioned at the base of the drill hole. Subsequently, the fully encapsulated rock bolts were developed to bind the bolt and surrounding rock after installation by means of resin or grout. The capability of load transfer of an encapsulated rock bolt is influenced by the resin or grout mechanical properties.

Aziz *et al.*, (2013*a*, 2013*b* and 2014*a*) carried out a detailed research study with the aim of establishing a general practice standard for determination of mechanical properties of resin. The study included; determination of the Uniaxial Compressive Strength (UCS), the Elastic modulus (E) value in compression, shear strength and rheological properties. These mechanical properties were examined at the University of Wollongong laboratory in relation to resin sample shape, size, height to width or diameter ratio, resin type, resin age and cure time. The following conclusions were reported:

- The UCS values determined from various shaped samples differed with respect to the sample shape and size and height to diameter ratio,
- Typically, the UCS values were highest for 40 mm cubes and 40 mm diameter cylindrical sample with height to diameter ratio of two,
- The ratio between cube strength and cylinder strength varied from 1.1 to 1.3,
- The E value increased as the resin sample curing time increased from 7 to 21 days,
- The cube samples exhibited higher E values in comparison with cylindrical specimens at various curing time,
- Similar to UCS values, the average shear strength of grout samples increased with increasing curing time,
- Cube samples were suggested as a universal shape for testing resin products as they can be easily prepared and tested.

A comprehensive report on the above study was further published by Aziz *et al.,* (2014*b*) through the Australian Coal Association Research Scheme (ACARP) organisation.

Hagan and Chen (2015) investigated UCS values of cube and cylindrical grout samples at different water to cement ratio, and it was found that:

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- Cube samples provided higher UCS values when compared to cylindrical specimens, and
- Strength of the material varied with water to cement ratio, showing a reduction trend with increase in the water quantity.

Recently, Mirza *et al.*, (2016) compared UCS values, E modulus and creep of two commonly used grout products (Stratabinder and BU 100 grouts) in Australian coal mining industry. It was reported that:

- Stratabinder HS grout was marginally better than the BU 100 grout for curing time of more than one day. For one day of curing time however, BU 100 samples showed better performance,
- Experiments indicated lower elastic modulus values for BU 100 when compared to Stratabinder HS under compressive cyclic loading,
- The elastic modulus determined by testing the samples using the Instron machine may have been influenced by the pronounced sample end effect, giving non-realistic low values,
- BU 100 showed higher creep value under a compression load of 100 kN for the duration of 15 min compared with Stratabinder HS,
- The difference between creep values of BU 100 and Stratabinder HS products was not significant. Both products suit equally for cable bolt installation in rocks for strata reinforcement.

This paper is a companion paper to the one recently published by Mirza *et al.*, (2016) and investigates following items that had not been studied previously:

- Uniaxial compressive strength (UCS) for a range of grout samples with various water to grout ratios,
- Shear strength for a range of grout samples with various water to grout ratios, and
- Perform triaxial testing of specific grout samples to analyse the effect of confining pressures, and obtain values for cohesion and internal friction angle.

Two commonly used grout products namely, Stratabinder and BU100 were used to cast samples (Figure 1).



Figure 1: a) Stratabinder HS b) BU100

EFFECT OF WATER TO CEMENT RATIO ON UCS

The procedure for sample preparation and testing for determination of UCS were the same as discussed by Mirza *et al.*, (2016). The curing time for samples prior to testing was seven days. Table 1 shows water to grout ratios that were used to cast samples. As the water to grout ratio increased, the sample mixtures contained more water per unit weight of grout. Mix 'A' contained the industry recommendation for the water to grout ratio for both Stratabinder HS and BU-100.

Water-Grout Ratios										
		Stratabinder HS		BU-100						
Mix ID	Litres/Bag	Water per 100g	Water:Grout	Litres/Bag	Water per 100g	Water:Grout				
1	5.6	28ml	0.28:1	3.6	18ml	0.18:1				
2	6.4	32ml	0.32:1	4.4	22ml	0.22:1				
А	7	35ml	0.35:1	5	25ml	0.25:1				
3	7.6	38ml	0.38.1	5.6	28ml	0.28:1				
4	8.4	42ml	0.42:1	6.4	32ml	0.32:1				
5	9	45ml	0.45:1	7	35ml	0.35:1				

Table 1: Water to grout ratios

Figure 2 shows a comparative chart showing the differences in Stratabinder and BU-100 uniaxial compressive strength for various water to grout ratios. The industry recommended water to grout ratio for each type of grout were used as a basis for comparison. Under these test conditions, Stratabinder HS was approximately 18% stronger than BU-100. It is clear from the position of the two curves that Stratabinder HS (shown in blue) operates at a higher water to grout ratio, considering the differences in peak values. This signifies that for any compressive strength value, the BU-100 mixture would need an 8%-12% reduction in water to match the strength of Stratabinder. The general shape and perpendicular deviation between each curve remains constant, which further demonstrates the homogenous effects of water content on the compressive strength and the ratio of water to grout type. There was an obvious correlation between compressive strength and the ratio of water to grout in the mixture. Adding water increased the space between grout particles, which prevented the formation of strong, close-knit bonds. As the sample cured, excess water evaporated, leaving pores of air throughout the sample. These pores offered no structural support and therefore contributed to the lower strength that observed. Results obtained during this testing coincided with the expected outcomes; showing a decreasing trend in UCS with increase in water to cement ratios.



Figure 2: UCS values for different water to grout ratios

EFFECT OF WATER TO CEMENT RATIO ON SHEAR STRENGTH

Punch shear tests were carried out on grout samples to determine the shear strength for various water to grout ratios. Samples were cast and tested following the procedure reported by Aziz *et al.,* (2014*b*). Samples were allowed to cure for seven days prior to testing. A typical disc sample and punch shear instrument that were used as part of this study are shown in Figure 3.



Figure 3: a) Typical disc sample b) Punch shear apparatus

Results from the punch shear test of Stratabinder HS and BU-100 are shown in Figures 4 and 5, respectively. Punch shear testing of Stratabinder HS samples provided inconclusive results, due to the broad spread of collected data while a general decreasing trend line is observed with increase in water to grout ratio. The relationship observed between strength and water to grout ratio corresponds with uniaxial compressive strength tests. It is inferred from the punch shear tests performed on BU-100 grout samples that increased water content subsequently decreased the grout shear strength. The shear strength values for BU100 samples ranged from 12.35 MPa to 9.01 MPa across an array of water to grout ratios.



Figure 4: Stratabinder shear strength against water to grout ratio


Figure 5: BU100 shear strength against water to grout ratio

In general, it was observed that higher water to grout ratio in the mixture causes a reduction in the shear strength. The mechanism behind this relationship corresponds to the particle structure of the samples, which was previously discussed. However, changing the water to grout ratio had a less effect on the shear strength in comparison with the UCS. Over the range of water to grout ratio, the shear strength of grout samples decreased by 27%, in comparison with UCS result's which decreased by 43%.

TRIAXIAL TESTING ON GROUT SAMPLES

Triaxial tests were conducted to determine internal friction angle (ϕ), cohesion (c) and tensile strength of grout samples. Grout samples were prepared on 100mm long cylinder moulds using a PVC pipe with a diameter of 50mm using mixture ID of A (Table 1). Samples were cured for seven days prior to testing in triaxial cell as shown in Figure 6. Three values of confining pressure including 2, 4 and 6 MPa were selected for testing. Each test was repeated three times and the average value was taken into account to calculate mechanical properties.



Figure 6: a) PVC mould containing prepared sample b) Triaxial cell in testing arrangement

The stress states is graphically represented in Figures 7 (Stratabinder) and 8 (BU100) by blue, red, and green arcs, which correspond to 2 MPa, 4 MPa and 6 MPa of confining pressure, respectively. The peak axial load and confining pressure for each set of samples are denoted by two intersection points on the x-axis. Mohr's Envelope was established and incorporated to calculate values for cohesive strength and internal friction angle. Values for cohesion and internal friction angle for Stratabinder samples were computed to range from 7.0 to 70 MPa and 50 to 70° respectively. These values for BU100 samples were 7.5 MPa and 52.76°. The data shown in Figure 7 suggests that Stratabinder cylinders exhibited a tensile strength of -5.5 MPa and for BU-100 cylinders the tensile strength was -5.05 MPa (Figure 8).



Figure 7: Mohr's envelope for Stratabinder



Figure 8: Mohr's envelope for BU100

CONCLUSION

Results of a systematic experimental study on grout samples cast using Stratabinder and BU100 were presented in this paper. Following main conclusions are drawn from this investigation:

- Water to grout ratio was a significant factor, which influences both the uniaxial compressive strength and the shear strength of grout,
- The UCS and he shear strength of grout samples decreased as water to grout ratio increased,
- Stratabinder and BU-100 had a cohesion of 7.70 MPa and 7.50 MPa, respectively, which showed a negligible difference, and
- The friction angle of Stratabinder ranged 50.70° as opposed to 52.76° for BU-100, these differences were relatively in significant difference.

Further shear tests on BU100 grout samples are recommended to increase the precision of collected data.

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VARIATION IN ANCHORAGE PERFORMANCE OF A HIGH CAPACITY MODIFIED BULB CABLE BOLT UNDER DIFFERING CONDITIONS IN A WEAK CONFINING MEDIUM

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ABSTRACT: Cable bolts are increasingly being used for ground reinforcement due to their high load carrying capacity, tendon length and flexibility that affords easy installation. There are a variety of cable bolt designs on the market and while many have been tested, few studies have examined anchorage performance in a weak confining medium that approximates behaviour in coal. This paper presents the results of testing a high capacity cable bolt, the MW9 indented cable bolt, in weak material of differing borehole diameters and grout strengths. Both the standard recommended borehole diameter of 42 mm and an oversize borehole of 52 mm were examined in combination with the standard high strength grout strength of 80 MPa and a lower strength of 62 MPa. In weak grout, borehole diameter had minimal effect on the peak load carrying capacity with only a 6% reduction in capacity from an increase in borehole diameter from 42 mm to 52 mm. However, with the strong grout, there was a 7% increase in peak load carrying capacity with borehole diameter. In the standard borehole diameter, increasing grout strength reduced the peak load carrying capacity by 4% whereas there was a 10% in the oversized borehole. In the vast majority of tests, failure occurred at the bolt/grout interface.

INTRODUCTION

Cablebolts usually consist of multi-wire strands that provide additional flexibility over traditional solid bar rockbolts. Commonly one or more steel cablebolt strands are inserted into boreholes of varying diameter and grouted using either a cement-based or resin material (Hutchinson and Falmagne, 1999). Cablebolts are considered as versatile, easy to install and can be installed from spaces with limited headroom (Hutchinson, 1992).

One of the major developments in recent times has been the development of modified cablebolts such as the bulbed cables. The Megabolt MW9 is one type of modified bulb cablebolt that comes in lengths ranging from 4 m to 11 m with a load carrying capacity from 60 tonnes to 80 tonnes (Megabolt, 2016). Even with the development of these high capacity cablebolts, failure of the cablebolts and/or ground still occurs. As reported by Hutchinson and Diederichs (1996), there are five types of failure modes that can occur in a cablebolt system. These include:

- Failure at cable and grout interface,
- Failure of surrounding rock mass,
- Failure of the grout,
- Failure of grout and rock interface, and
- Rupture of the wire of the cable bolt.

Laboratory pullout tests are often used to assess the load carrying capacity of a cablebolt. The testing apparatus is used to simulate *in situ* conditions in the laboratory. Various advancements have been made in the area of laboratory pullout testing apparatus. The initial split-pull testing apparatus developed by Fuller and Cox (1975), consisted of grout and cablebolts confined in steel split pipes. However, it was found that the properties and confinement provided by a steel tube is different to that

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experienced in field conditions and consequently influenced anchorage response and behaviour (Hagan *et al.*, 2015).

Since 1975 various modifications have been made to the initial split-pull testing apparatus. The latest development in the area of pullout testing has been the development of UNSW modified laboratory short encapsulation pull-test (LSEPT) as reported by Chen *et al.* (2015). This new modified axial testing facility as shown in Figure 1 was used to analyse the performance of MW9 bulbed Megabolt.



Figure 1: Front and side view of the new axial testing facility (Chen et al., 2015)

The material properties of the grout used to anchor the cablebolt and the borehole diameter impact the performance of cablebolts. Robinson and Sharrock (2010) investigated the effect of borehole diameter on the pullout capacity of the cablebolt using the standard single embedment test. This involved borehole diameters of 42 mm, 52 mm, 81 mm and 106 mm. They found that when used with a cement grout that diameter had little effect on the peak pullout load of the cablebolt whereas water content in the grout mixture ratio and bulb frequency had a more dominant role on cablebolt behaviour. They varied the water to cement ratio between 0.35 and 0.45 and found that with a 0.35 W:C ratio more samples failed via rupture than with a 0.45 W:C sample. A similar trend was observed by Hyett *et al.* (1992).Chen *et al.* (2015) reported on the development of a modified version of the laboratory short encapsulation test (LSEPT), a new axial testing facility capable of assessing the performance of modified cablebolts. As part of this project, a study was undertaken on the effect of borehole diameter on the performance of MW9 Megabolt. The larger borehole diameter achieved a near 25% larger peak load than the standard borehole diameter as shown in Figure 2.



Figure 2: Effect of borehole size on load carrying capacity of MW9 (Hagan and Chen, 2015)

METHODOLOGY

Laboratory testing was carried out using the newly modified LSEPT axial testing facility at The School of Mining Engineering UNSW.

Sample preparation

A total of 20 low strength test samples were prepared as the confining medium in which a cablebolt was embedded. The test samples were prepared from a cement-based material cast into 300 mm diameter cylindrical moulds. Rajaie (1990) first studied the effect of sample diameter size on the pull out strength of plain strand cable bolts. The study found that there was minimal change in the load carrying capacity of the bolts for sample diameter within the range of 200 – 300 mm. Recent study by Ur-Rahman, Hagan and Chen (2015) observed the same trend as reported by Rajaie (1990).

In preparing the moulds, PVC pipes were used to create varying boreholes with diameters of 42 mm and 52 mm. The PVC pipes were wrapped with plastic tube to create a constant manufactured rifled effect in the borehole as is shown in Figure 3.



Figure 3: PVC pipe wrapped with plastic tube

Sample moulds were fabricated from Ezytube[™] fixed onto fibre board sheets in the centre of which was fixed a PVC pipe as shown in Figure 4. A single batch of low strength concrete was then poured into the 20 moulds. The material had a measured UCS of 10.9 MPa.



Figure 4: Prepared moulds to create the test samples (left) and pouring of cement mixture into moulds (right)

After 24 h, the PVC pipe and plastic tube was removed and the cardboard mould discarded. The test sample confining medium was left to cure for a minimum of 28 days. Lengths of MW9 indented cablebolt was prepared with the lower 90 mm wrapped in PVC heat shrink tube, this was done to ensure a constant embedment length during the pullout test. The cablebolt was then grouted into each confining medium. Ten samples had a grout strength of 80 MPa, the recommended grout strength and, a further ten samples had a strength of 62 MPa representing a grout mixture with a higher water content. Anchor tubes were then installed and grouted over the remaining exposed cablebolt above the confining medium. Anchor tubes are used to stop the cable bolt from unwinding during the pullout process and secure the free section of the cable bolt (Hagan and Chen, 2015). Anchor tube has a key slot with a locking key as shown in Figure 5 which prevents the whole section of the cable bolt from unwinding during a test.



Figure 5: Anchor tube, bearing plate with locking key that prevented rotation during a test (Hagan and Chen, 2015)

Test setup

Each test arrangement involved placing a paired set of split steel tubes around the confining medium. Prior to a test, the bolts on the split tubes were tightened to a constant torque of 50 N.m and the test sample assembly was ready for testing. Laboratory testing was carried out in the newly modified axial testing facility at UNSW that is suited to the testing of a range of modified cablebolt designs. The

testing apparatus consists of double acting hydraulic cylinder which provided the axial pull-out force at a displacement rate of 0.27 mm/sec.

RESULTS AND ANALYSIS

A total of 20 test samples were used to assess the variation in axial load carrying capacity of MW9 indented cable bolt under:

- 1. varying borehole diameter; and
- 2. varying strength of the grout used to anchor the cable bolt.

Borehole diameter of 42 mm and 52 mm was chosen to assess the axial load carrying capacity of the Megabolt MW9 indented cable bolt. 42 mm represents a standard borehole diameter as recommended by the cablebolt manufacturer. A 10 mm oversize borehole diameter was also chosen. Cablebolt grout strengths of 62 MPa and 80 MPa were used. In total four parameters were varied, each with five samples. 15 samples were tested at a confining torque of 50 Nm. The remaining samples were tested at a confining torque of 40 N.m. Samples tested at 40 Nm confining torque was as follow:

- two samples from 52 mm borehole size with grout strength of 62 MPa;
- two samples from 42 mm borehole size with grout strength of 62 MPa; and
- one sample from 42 mm borehole size with grout strength of 80 MPa.

Analysis of borehole diameter

Standard borehole diameter of 42 mm and oversized borehole diameter 52 mm were used to analyse the effect of changes in borehole diameter on the load carrying capacity of MW9 indented cable bolt system.

62 MPa (weak) grout strength samples

Ten samples were tested at the grout strength of 62 MPa. For analysis purposes, three of the most consistent results with confining torque of 50 Nm were chosen. These results are shown in Figure 6 for 42 mm and 52 mm borehole size respectively.



Figure 6: Results of testing with a 62 MPa grout in a 42 mm (left) and 52 mm (right) diameter borehole

The graphs indicate little substantial difference in performance between the two borehole diameters, which indicates that the performance is not sensitive to a small change in borehole diameter in weak grout. Residual stress follows the same pattern with a reduction in load post-peak. However there

does appear to be greater evidence of cyclic "slip-lock" behaviour in the smaller diameter borehole. Table 1 shows the 42 mm borehole diameter had on average a 6% higher load carrying capacity than the 52 mm borehole. As the borehole size was increased, peak load carrying capacity decreased by 6%. Maximum peak load was 6 % lower as the borehole size was increased from 42 mm to 52 mm.

Table 1: Performance parameters for the cablebolt for varying borehole diameter in 62 MPa grout

	42 mm borehole	52 mm borehole	Difference
Average Peak Load (kN)	206	193	-6%
Maximum Peak Load (kN)	213	200	-6%
Average initial stiffness (kN/mm)	65.0	47.6	-27%
Maximum initial stiffness (kN/mm)	72.2	61.9	-14%

In terms of stiffness of the support system, an increase in borehole diameter size resulted in 27% reduction in the average initial stiffness. Maximum initial stiffness also fell by 14 % as the borehole size was increased. In the majority of cases, failure occurred at the grout/cablebolt interface and was consistent across both 42 mm and 52 mm samples as shown in Figure 7.





Figure 7: Dominant failure mode was between the grout and cable bolt (left) except in one 52 mm sample where failure occurred at the rock/grout interface (right)

80 MPa (strong) grout strength samples

Borehole diameter was varied in both strong and weak grout. Figure 8 shows graphs of the results for the 42 mm and 52 mm borehole diameters respectively.



Figure 8: Results of testing with an 80 MPa grout in a 42 mm (left) and 52 mm (right) diameter borehole

Comparing the two graphs in Figure 8, it can be noted that load carrying behaviour for MW9 indented cable bolt was again consistent between the two borehole diameters in strong grout. Slip/lock behaviour was more prominent in the stronger grout. Table 2 shows the trend was reversed with the stronger grout with a 7% increase in average peak load with an increase in borehole diameter. Maximum peak load was also 6% higher in the larger borehole. This could be attributed to better bond strength offered by strong grout. Interestingly, the peak loads were the same for both grout strengths.

	42 mm borehole	52 mm borehole	Difference
Average Peak Load (kN)	199	212	+7%
Maximum Peak Load (kN)	204	216	+6%
Average initial stiffness (kN/mm)	64.9	55.7	-14%
Maximum initial stiffness (kN/mm)	95.7	62.8	-34%

Table 2: Performance parameters for the cablebolt for varying borehole diameter in 80 MPa grout

There was again a reduction in the system stiffness of 14% as the borehole size was increased in strong grout. Initial stiffness values for 52 mm samples were more consistent than 42 mm samples. This is due to the difference in failure modes between 42 mm and 52 mm samples. All three samples with 52 mm had failure at the grout/cablebolt interface. Whereas in the smaller borehole, different failure modes were observed in the three test samples, these being failure at the:

- rock/grout interface;
- bolt/grout interface; and
- combination of bolt/grout and rock/grout interface.

Increasing borehole diameter in both weak and strong grout had a negative impact on the initial stiffness for indented MW9 cable bolts.

Analysis of grout strength to anchor the cable bolts

Grout strength is another important parameter that can affect the load carrying capacity of the MW9 indented cablebolt. A weak grout with strength of 62 MPa and strong grout with 80 MPa were used to analyse the effect of the change in grout strength on the load carrying capacity of the MW9 indented cablebolt system.

42 mm (standard) borehole diameter samples

Graphs of the performance curves for 62 MPa and 80 MPa grout strength are shown in Figure 9. Comparing the graphs in Figure 9 it can be noted that there is minimal change in the peak load carrying capacity of MW9 indented cable bolt as the grout strength is increased in standard borehole samples. Residual stress follows a similar profile in both scenarios with a sharp reduction in load carrying capacity post-peak. This reduction is followed by a gradual increase in the load followed by a gradual decline near the end of sample testing.

Table 3 summarises the change in peak load as the grout strength is varied for the standard borehole diameter samples.



Figure 9: Results of testing in standard borehole of 42 mm with a 62 MPa grout (left) and 80 MPa grout (right)

Table 3: Performance parameters for the cablebolt for varying grout strength in standard42 mm borehole

	62 MPa grout	80 MPa grout	Difference
Average Peak Load (kN)	206	199	-4%
Maximum Peak Load (kN)	213	204	-5%
Average initial stiffness (kN/mm)	65.0	64.9	0
Maximum initial stiffness (kN/mm)	72.2	95.7	+33%

The peak load carrying capacity for strong grout was 4% less than that of weak grout in standard borehole diameter. Maximum peak load was also 5% lower in the strong grout samples when compared with weak grout. Although the peak load carrying capacity decreased for the strong grout, the change was minimal. These results are contrary to the study done by Robinson and Sharrock (2010), which concluded that higher grout strength results in higher pull out load.

In terms of stiffness, the average initial stiffness was observed to be similar in both grout samples. However, in the 62 MPa grout samples, the values for initial stiffness were constant and less variable when compared with 80 MPa grout samples. This is due to the difference in failure modes. In 62 MPa grout samples the failure was between the bolt and the grout interface for all the 3 samples. Variable failure modes were observed in 80 MPa grout samples. These consisted of failure at:

- rock/grout interface;
- bolt/grout interface; and

52 mm (oversized) borehole diameter samples

Graphs comparing the results for the 62 MPa and 80 MPa grout strength in the oversized borehole are shown in Figure 10.



Figure 10: Results of testing in oversized borehole of 52 mm with a 62 MPa grout (left) and 80 MPa grout (right)

A higher peak load carrying capacity was observed in the oversized borehole samples. This is opposite to the results obtained from varying grout strength in standard borehole samples. Table 4 summarises the comparison in peak load for varying grout strength in 52 mm diameter samples.

Table 4: Performance parameters for the cablebolt for varying grout strength in oversized52 mm borehole

	62 MPa grout	80 MPa grout	Difference
Average Peak Load (kN)	193	212	+10%
Maximum Peak Load (kN)	200	216	+8%
Average initial stiffness (kN/mm)	47.6	55.7	+17
Maximum initial stiffness (kN/mm)	61.9	62.8	+1

Comparing strong and weak grout in oversized borehole it was found that the stronger grout achieved a 10% higher average peak load than that of weak grout. This is contrary to the results achieved in standard borehole size where average peak load was decreased by 4% in strong grout. Results achieved from varying grout strength in oversized borehole are more in line with those of Robinson and Sharrock (2010).

Average initial stiffness increased by 17% for strong grout in oversized borehole diameter samples. This is different to the results obtained from standard borehole size where initial stiffness largely remained unchanged for varying grout strength. Failure mode occurred between bolt and grout interface across weak and strong grout samples. However, one sample was observed to have failed between rock and grout interface in weak grout.

Analysis of confining torque

A number of tests were undertaken to assess the impact of varying the initial torque loading of the bolts used in the split tubes. A total of five samples were tested at a lower torque of 40 Nm compared to the standard 50 Nm used in the majority of tests. A reduction of 13% was observed with the lower torque in the average peak load with the 52 mm borehole with 62 MPa grout. However the initial stiffness was 20% higher. This indicates that the level of confinement through tightening of the bolts is an important factor in determining the performance of MW9 indented cable bolt system. A similar trend was observed with smaller 42 mm borehole and 62 MPa grout strength. 40 Nm confining torque sample resulted in a reduction in the average peak load of 15%.

CONCLUSION

A study was undertaken to analyse the impact of varying grout strength and borehole diameter on the performance of MW9 indented cable bolt in weak confining medium. The effect of a change in the torque of the bolt assembly was also investigated.

Borehole diameter was found to have minimal impact on the peak load carrying capacity of MW9 indented cable bolt in weak grout. Increasing borehole diameter from 42 to 52 mm led to a 6% reduction in peak load carrying capacity. In strong grout, the opposite effect was observed with a 7% higher peak load in the larger borehole. While the differences are not significant, it can be concluded that when using a strong grout, greater benefit can be gained with an oversized borehole. Increasing borehole diameter in both standard and oversized borehole samples decreased the average initial stiffness. Residual load followed the same with cyclic slip/lock behaviour with the borehole regardless of the borehole diameter.

Increasing grout strength in standard borehole reduced the peak load carrying capacity by 4%. However, increasing the grout strength in oversized borehole led to 10% increase in peak load carrying capacity. Therefore, the study confirmed that oversized borehole diameter should be used for strong grout to improve the peak load carrying capacity of the MW9 indented cable bolts.

A number of tests were undertaken at a lower bolt torque of 40 N.m. The reduction in confining torque led to a significant reduction in peak load carrying capacity for MW9 indented cable bolt. In the majority of cases, the failure mode in the test samples was at the bolt/grout interface.

The study offers insights into the effect of varying borehole diameter, grout strength and confining torque on the performance of MW9 indented cable bolts. Results achieved are consistent and reliable. These results will help the industry make informed decisions in relation to optimum selection of ground reinforcement systems. This will lead to improve safety standards in civil and mining industry.

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SINGLE SHEAR TESTING OF VARIOUS CABLE BOLTS USED IN AUSTRALIAN MINES

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ABSTRACT: Sixteen single shear tests were carried out on eight geometric cable variations provided for testing from Australian suppliers – Jennmar, Megabolt and Minova. Each test was subjected to varying pre-tension values of zero and 15 tonnes, exploring the effect of plain, spiral, bulbed, indented and a combination of plain and indented wire strands. The results obtained demonstrated that the shear strength of plain strand cable was higher than the spiral and/or indented profiled cables with direct correlation to the strands ultimate tensile strength. All the plain profiled cables experienced an element of partial debonding suggesting that their application at embedment length less than 1.8 m each anchor side may not be adequate. The spiral and indented profile strands provided greater bond strength at the cable-grout interface due to the surface roughness of the wires imposing an interlocking effect, leading to reduced shear displacement. The data suggests that the spiral profile was superior to the indented profile due possibly to the compromised integrity of the strand from the impact of stress raisers when creating the indented profile. No study was carried on the button indented profile cable bolts. This report is the first validation that type of apparatus selected to test the shearing capacity of a cable strand will not affect results.

INTRODUCTION

The practice of utilising cable bolts as a means of secondary support when the bolted height does not provide sufficient support has become an industry standard. Since the introduction of this ancillary support method, studies regarding the loading mechanisms of cable bolts have increased, particularly in the form of axial loading. Direct shear loading is still in its infancy with the majority of the research undertaken at the University of Wollongong (UOW). This paper continues and extends the work of the UOW Rock Bolting and Strata Control Research team by extending the scope of the studies to include single shear testing.

The new shearing apparatus was designed and constructed by Megabolt, a strata control product manufacturer, in response to the deficiencies of the current industry standard for the single shear testing of cable bolts outlined in BS7861-2:2009. The new methodology as designed allows for active reinforcement of the system and full encapsulation in a brittle host material allowing for the rock-grout interface to be assessed, creating more realistic results. The drawback of this methodology is the intense sample preparation required compared to preceding studies, which will impact for further studies to replicate the conditions of the test.

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Ground reinforcement can be classified in two distinct ways. Reinforcement is deemed primary if it is installed during the excavation sequence, whereas a support is secondary if it is installed sometime after the formation of the roadway. Conversely, ground support can be classified in terms of the active or passive support it imposes on the dynamic system. Active support applies a force to the rock mass modifying the mechanical behaviour to minimise displacement, particularly in jointed and loose rock units. Whereas, a passive ground support mechanism does not impose any initial force, rather it provides a resistive force as the rock deforms over time. Generally, rock bolts will be installed immediately post the creation of the excavation, followed, when necessary by the complementary ground support mechanism of cable bolts. The secondary support is utilised when the bolted height does not provide sufficient support, connecting the fractured zone to a more competent strata layer. Cable bolts consist of high strength steel wires coiled into a strand, which is installed into drilled holes and bonded to the rock mass by grout. The application of cable bolts, in conjunction with primary ground supports and a confining medium such as rock bolts and mesh, is presently a common industry standard for an integral excavation. The main function of cable bolts is to stabilise and strengthen rock mass, as well as provide resistance to bed separation and regulate post-failure deformation (Galvin 2016). Understanding the performance of cable bolts is becoming increasingly important to reduce expenditure without reducing its effectiveness in redistributing stress.

The two primary forces that underground support systems are subject to are axial loading and shear loading. Over the past 40 years, research has commenced into stimulating these loads to select the most appropriate cable for specific overlying and surrounding rock masses (Thomas 2012). Globally, the majority of the published research studies surrounding the performance of cable bolts is centered around axial loading of the cable bolt known as 'pull testing'. Data from pull testing is widespread as it can be conducted in the laboratory and the field due to its relatively simple testing method. Methods surrounding applying a direct shear load are still in its infancy due to its complexity and specialist laboratory process (Hutchinson and Diederichs 1996). Research reported in this paper is initiated with ACARP funding (project C24012), which focuses on cables bolts used in Australian mines and conditions.

PROCEDURE

Equipment

The single shear apparatus is a horizontally aligned integrated system, consisting of the shearing rig and a 120 t compression machine, as shown in Figure 1. The shearing cylinder is fabricated in two sections, each containing 1.8 m of concrete anchor cylinder, providing a centrally located shearing plane. The shearing cylinder is enclosed in steel clamps to provide confinement during shearing. The shear load is applied by four hydraulic rams, located at the bottom of the shear rig, with the applied shear load measured by a pressure transducer and analogue gauge.



Figure 1: Single shear test apparatus

The hydraulic pressure originates from either a hand pump or power pack of suitable capacity. The hydraulic pressure is fed to a manifold which distributes the pressure to the compression testing machine legs. A pressure transducer in conjunction with an analogue pressure gauge monitors the pressure in the manifold. The rate of loading is applied through manual application, with an aim to apply a constant load in line with F 432-04 (ASTM 2005) and BS7861-part 2 of between < 4 mm mm/min. Linear Variable Displacement Transducers (LVDTs) were utilised to measure shear displacement at the shearing plane and any debonding at the cable ends. A datataker is used to record the readings of the pressure transducer and the LVDTs at a constant time intervals, which are utilised for further data analysis.

Sample preparation

Each cable bolt was installed in 250 mm diameter 3.6 m long and concrete cylinders. Cardboard cylinders were used as mould to make the test samples. Each cylinder consisted of a length and diameter of 900 mm and 250 mm accordingly. The axially laid central borehole within the sample was created through a 1000 mm steel rod with 8 mm dimeter plastic conduit wrapped around the circumference of the rod to simulate rifling. The 40 MPa concrete with 10mm aggregate was prepared by an external body, to ensure that the concrete met requirements, slump tests were conducted on the fresh concrete to ensure integrity in relation to moisture and consistency of the mix. Once the concrete set the steel rod and plastic conduit were removed from each cylinder and then the cardboard mould was cut off. The concrete cured for a minimum of 28 days to allow for the nominal strength to be achieved. Each single shear test required four concrete cylinders with two sets of concrete cylinders fixed together to create the 1800 mm anchor cylinders on either side of the shear interface.

The frictionless shear interface was created through the use of two Teflon plates which had a thickness of 2 mm allowing for 4 mm opening between the concrete anchor cylidners to remove the fricitional effects of the concrete contact at the interface as shown in Figure 2. Neoprene seals were also utilised to ensure the grout would not percolate from the annulus. The adhesion of the Neoprene seals and teflon plates to both of the cylinders occurred simultaneously with the second concrete anchor cylinder positioned in the frame after two minutes of curing time. Once the grout adaptor plate was glued to the other extremity of the 3600 mm span, the primary clamp was enclosed and fastened with bolts.



Figure 2: Teflon sheets with Neoprene seal for frictionless shear of the cable

Before grouting occurred, the pre-tensioning and grouting frame was positioned at an angle of 65° to allow for the bottom-up grouting method. This grouting method is where grout is propelled from the lower extremity of the cable filling the entire annulus area. Moreover, the angle allows for gravity to provide a weight force to prevent any air bubbles forming. Figure 3 shows the assembled cable bolt being grouted in the concrete cylinders and left to cure over a minimum period of one month. Stratabinder HS grout was used to encapsulate the cables in the concrete cylinders of all the cables used in the study. Once grouted the samples were left to cure for a minimum of 28 days prior testing.

Methodology

Once the mandatory time for curing was reached, each sample was disassembled from the frame and mounted to the shearing rig. When the sample was correctly positioned and fastened in the shearing rig, steel clamps, placed around the concrete blocks to provide a confining pressure to the sample. The action of applying a confining medium provided a more accurate replication of *in situ* conditions of the force applied to the cable from adjacent strata.



Figure 3: Angled cable grouted samples in 3.6 m concrete cylinder

Linear Variable Deferential Transducers (LVDTs) were used to monitore displacement during cable shearing process as well as debonding. As shown in Figure 4, two LVDTs were mounted on both ends of the concrete cylinder to provide a numerical value for the cable axial displacement, when sizeable displacements affect the functioning of the strain gauges. Also, the extremity LVDTs provide information about the possible debonding of the cable. The pressure transducer, LVDT at the shear interface and the strain gauges were all connected to the data logger to monitor and record the data at a specified time interval. The use of stain gauges were soon abandoned in favour of LVDTs. A hydraulic power pack was connected to the hydraulic rams for vertical shearing. The pressure was applied manually to allow a loading rate of < 4 mm/min.





Figure 4: An LVDT monitoring displacement at cable end with and without strain gauges

RESULTS AND DISCUSSION

Table 1 provides a summary of the data obtained from all of the sixteen single shear tests. The table presents the peak shear load of each cable with the corresponding shear displacement. Figure 5 shows photos of few sheared cables strands and Figure 6 shows profiles of different cable shear load-shear displacement of 15 test results. All the plain MW10 samples as well as SUMO cables experienced some element of debonding, differentiating it from other rough surface wires of spiral and indented cable samples. It is clear that the plain wire configuration reduces the bond strength at the cable-grout interface due to lack of surface roughness of the wires. Strain gauges were used to monitor axial displacement in the first six cables. The strain gauge and LVDT readings for the MW10 samples are shown in Figure 4, depicting debonding at certain displacements. Debonding occurred also in plain superstrand cable bolt (test 15). It is suggested that by using silastic to glue strain wires on the cable surface may have contributed to debonding of MW10 cables, however, both SUMO cables and superstrand cables were debonded with neither cables being coated with silicon glue on the plain wire surface. According to McKenzie (2014) some elements of partial debonding was observed in their past tests. This aspect of the study is being considered for further study.

		Cable bolt properties							
Test	Product Name	Cable cross- section	Strand UTS (t)	Cable geometry	Pretension load (t)	Peak Shear load (t)	Displacement at peak (mm)	Cable debonding	Peak Shear Load / UTS (%)
1	MW10	Plain	70	No bulbs	15	68.3	93.3	Yes	97.6
2	MW10	Plain	70	6 bulbs	0	63.8	62.6	Yes	91.1
3	MW10	Plain	70	6 bulbs	15	60.4	56.0	Yes	86.3
4	MW9	Spiral	62	6 bulbs	0	47.7	43.5	No	76.9
5	MW9	Spiral	62	6 bulbs	15	43.1	47.4	No	69.9
6	MW9	Spiral	62	No bulbs	15	49.7	41.7	No	67.3
7	Secura HGC	Combination	68	6 bulbs	0	64.7	51.8	No	95.2
8	Secura HGC	Combination	68	6 bulbs	15	55.9	45.9	No	82.2
9	SUMO	Plain	65	6 bulbs	0	54.7	71.8	Yes	86.8
10	SUMO	Plain	65	6 bulbs	15	67.1	78.2	Yes	106.5
11	ID-SUMO	Indented	63	6 bulbs	0	46.4	46.9	No	73.7
12	ID-SUMO	Indented	63	6 bulbs	15	37.4	30.9	No	59.4
13	ID-TG	Indented	60	No bulbs	0	44.0	51.3	No	69.8
14	ID-TG	Indented	60	No bulbs	15	36.3	30.9	No	57.6
15	Superstrand	Plain	60	No bulbs	15	51.4	90.2	Yes	85.7
16	Garford	Plain	2 x 27	Bulbed	0	43.7	46.8	Yes	80.9

Table 1: Cable bolt properties and test r	results
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MW10

SUMO So Figure 5: Three shear tests for three different cables

Secura HGC

Tests 7 and 8 evaluated the shear loading of the Secura Hollow Groutable Cable (HGC) with and without the application of pre-tension. The Secura HGC consisted of four indented wires and five plain wires, surrounding the hollow central tube. The UTS of the Secura is 68 t, which is the second highest UTS strength of all the tested samples. Test 7 with zero pre-tension applied to the cable returned a peak shear load of 64.7 t and a shear displacement of 51.8 mm. Test 8 with 15 t pre-tension loads applied to the Secura HGC cable failed at shear load of 55.9 t and a displacement of 45.9 mm. Both the Secura bolt tests showed that the peak shear load decreased with increased pre-tension load. It is important to note that the Secura HGC did not undergo any displacement sourced from debonding as the profiled induced greater bond strength at the cable-grout interface due to surface irregularity. The study suggests that the combination of the two profiles, plain and indentation, appears to influence the cable anchorage performance.

The effect of increased pre-tension across nearly all the cables results in significant drops in shear load, except for the SUMO. Test 9 applied no pre-tension to the sample which recorded a peak shear load of 54.7 t with a shear displacement at peak load of 71.8 mm. Test 10 applied a 15 t pre-tension load resulting in the peak shear load of 67.1 t and a displacement at this load of 78.2 mm. The shear load of 67.1 t was above the expected Ultimate Tensile strength (UTS) of the plain sumo cable and the reason is unclear. In general, the pre-tensioned samples achieved a lower peak shear load, which does reflect on the hypothesis of stiffness reducing shear loads resisted.

It is worth mentioning that the general understanding of the shear strength of the steel being around 70 % of the ultimate tensile strength may not be applicable to cable strands when subjected to shearing. The cable strand is invariably consists of several wires as well as a grout tube filled with grout, which behave differently when sheared.

The effect of spiral versus smooth wire on shear load was indicated by comparing the MW9 and MW10, even though the MW10 has an additional wire in the cable bolt. The spiral wire MW9 achieved average shear loads of 75% x UTS, whereas the smooth wire MW10 achieved average shear loads of 92% x UTS. Naturally the effect of MW10 debonding has an influence on results, with excessive shear displacement.

The effect of indented versus smooth wire on shear load was indicated by comparing the SUMO and ID-SUMO. The indented wire ID-SUMO achieved average shear loads of 66% x UTS, whereas the smooth wire SUMO achieved an average of 96.7% x UTS.

The effect on shear strength from the presence of bulbs in the Megabolt cable bolts was assessed by comparing tests 1 and 3 on the MW10, and tests 5 and 6 on the MW9. An increase in shear load of 13 - 15% was found in the Megabolt cables by removing bulbs. When assessing the effect of bulbs in the indented ID-SUMO and ID-TG, it was found that removing the bulbs decreased shear strength by 3 - 5%. Note that both TG and SUMO cables are made from the same hollow strand, and that the ID-

SUMO indented hollow cable used in the study was only a trial batch that were made for test work and is not marketed in Australia.

The plain Superstrand cable bolt with 15 t pre-tension load achieved 52.40 t peak shear load at 90 mm shear displacement. The result from test 16 indicated that the twin wire, Garford, cable bolt with 0 t pore-tension load reached 44.55 t at 46.8 mm of shear displacement.

There was no difference in the failure loads between the plain and spiral wires of Megabolt cables MW9 and MW10. Two 500 mm long wires of plain and spiral weighed 150.232 gm for spiral and 150.302 gm for plain wires. The failure loads were 6.6 t for MW 9 spiral wire and 6.8 for MW10 plain wire, demonstrating no loss in weight and strength in two wire versions. This is in confirmation with inhouse results of failure load and elongation graphs observed from Megabolt internal test results, and are also evident from cross sectional photos of cut strands of MW9 and MW10 respectively as shown in McKenzie (2014). Secura HGC bolt indented wire lost around 10% of its strength and diameter as compared with plain wires. Similar weight and strength losses were observed in superstrand cable. Figure 7 shows the loss of strength in both Secura and Superstrand cables.



Figure 6: Shear force values Vs shear Displacement values for different tested cables



Figure 7: Tensile loads of both Secura HGC and Superstrand cable wires (Aziz, *et al*, 2010 and 2014)

CONCLUSION

The new shearing apparatus addresses the deficiencies of the current British Standard (BS7861-2:2009) for the single shear testing of cable bolts. Twelve tests on different cable bolts confimed that;

- The inverse relationship between increasing pre-tension load and the decrease in peak shear load and displacement.
- Cable bolts with rough wires result in a reduced shear load in comparison with cable bolts all smooth profiled wires.
- The effect of bulbing in some cables is inconclusive with both reduction and increases found in the comparison testing, however, bulbing may have influence on the integrity of the cable grout bonding.
- A cable bolt comprising a combination of smooth and spiral wires performed well in shear without debonding.
- Plain cables more readily debond at the cable-grout interface due to the smooth wire surfaces,
- The failure load difference between plain and profiled strands is proportional to the weight loss due to wire indentation. MW9 and MW10 strand wires were equal in weight

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DOUBLE SHEAR TESTING OF CABLE BOLTS WITH NO CONCRETE FACE CONTACTS

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ABSTRACT: A new series of double shear tests were carried out using a newly modified double shear apparatus which prevented contacts between concrete block surfaces during shearing. 13 double shear tests were carried out using 21 mm diameter 19 (9x9x1) seal construction wire strand cable (also called Superstrand cable), Plain SUMO, Indented SUMO, Spiral MW9 and Plain MW10 cable bolts. These cables were tested subjected to different pretension loads. Concrete blocks with Uniaxial Compressive Strength (UCS) of 40 MPa and Stratabinder grout were used for all the tests to maintain test consistency. The results show that the peak shear load and the corresponding shear displacement decrease by increasing the pretension load of the tested cable. The Ultimate tensile strength, lay length, number of wires and cable bolt surface profile type (plain and spiral/indented) are important factors in total shear strength of the cable bolt.

INTRODUCTION

Mining in Australia is one of the safest industries but there are still some fatal accidents. The number of deaths were10, 13 and 13 in 2013, 2014 and 2015 respectively (Safe work Australia, 2016). The current emphasis is to towards zero fatality Therefore, one of the greatest concerns to designers and engineers during and after excavation is the stability of underground excavations and surface mining slopes. It is important for designers to understand and have views on various forms of instability and the mechanisms of failures and associated conditions to support the unstable surfaces by installing various types of cable bolts for effective ground stabalisation.

The application of cable bolts as a secondary support system is a growing trend in underground coal mines worldwide (Fuller and O'Grady, 1993). The first type of cable bolts used in mines consisted of seven high tensile strength and pre-stressed wires, which were arranged in the strand with plastic spacers. The plain strand cable bolts with poor load transfer properties due to smooth and straight profile wires were initially introduced to mines, as a temporary means of rock reinforcement. Over the years, a number of modifications has been introduced to the plain strand cable, such as strand surface profiling and indentations (Schmuck, 1979), double plain strand (Matthews *et al.*, 1983), epoxy-coated strand (Dorsten *et al.*, 1984), fiberglass cable bolt (Mah, 1990), birdcage strand (Hutchins *et al.*, 1990), bulbed strand (Garford, 1990), and nutcage strand cable bolts (Hyett *et al.*, 1993). These various types of cable bolt have been incorporated to improve the load transfer capacity as permanent ground reinforcement.

While the Australian mines use a variety of cables to suit ground condition, the trend, in recent years in both Canada and in the USA has been to down size the cable dimensions to less than 20 mm diameter. According to Tadolini (2016), the most popular cables used in the USA are15 mm (0.6 inch) diameter strand (30 t capacity) and 18 mm (0.7 inch) diameter strand (nominal 40 t). They are both 7 wire strand (King-wire and 6 outer wrap wires). There are very few high strength strand cable bolts sold in the US or Canada, because of cost. In 2016, 1.32 million of 15.25 mm (0.6 in) diameter cable bolts and 25,000 of 18 mm (0.7 in) vertical cable bolts were sold in the US.

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Axial, shear or the combination of axial and shear failure are different mechanisms of failure of cable bolts. A combination of both axial and shear failures typically occurs *in situ*. Strength characteristics of cable bolts are important factors in the axial failure of cable bolt in comparison with the strength of grout and rock mass (Cao, 2012). When the strength of cable bolt is lower than the optimum required support, the cable bolt fails. The amount of shear in the bolt-grout surface is higher than the grout-rock surface because of smaller effective area. Pull out and shear tests (single and double shear tests) are the known methods of evaluating the performance of cable bolts. Pull testing has been conducted over the years by researchers to determine tensile failure and load transfer capacity of cable bolts and rock bolts (Hawkes and Evans 1951; Fuller and Cox 1975, Fuller and Cox 1978; Goris 1990; Yazici *et al.*, 1992; Hyett *et al.*, 1992; Diederichs *et al.*, 1993; Bouteldja 2000; Clifford *et. al.*, 2001; Morsy *et al.*, 2004; Thomas 2012; Chen *et. al.*, 2016).

15 single shear tests using various concrete grades, three steel sizes and four different angles for stirrup were conducted by Dulacska (1972) to assess bolt action in cracked concretes. Stillborg (1984) conducted a series of single shear tests to determine the shear behaviour of fully grouted cable bolts. It was found that when the angle of cable bolt to the joint surface is 45° compared with 90°, the shear resistance of the grouted cable bolt was significantly higher. Also, the maximum of shear resistance in the cable bolt occurred at 10 mm of shear displacement. Four modes of failure of a fully grouted cable bolt were reported by Thomas (2012) and include; failure at cable to grout interface, failure through grout column, failure at grout to rock interface and failure through rock around borehole wall. The first two failure modes are not common while the third and fourth failure modes are common; however, the cable bolt failure is the most common mode. Goris *et al.*, (1996) conducted a series of direct shear tests on 15.25 mm (0.6 in) diameter cable bolts with 26 mm hole diameter using concrete blocks with 69 MPa strength and the joint surface area of 0.078 m². The result from the single shear test was higher than the double shear test for the same type of cable bolt.

Craig and Aziz (2010) conducted a series of double shear tests on 28 mm TG hollow strand cable bolt. Cable bolts are under tensile and shear load in mines because of the roof deformation loads; therefore, two tests were conducted to investigate the shear behaviour of cable bolts in different displacement limitation and initial pretension loads. Developed in 2007, the Jennmar TG cable is a 618 kN (63 t) post grouted cable bolt with 9 wire strand (each element is 7 mm in diameter), which surrounds a 14 mm hollow steel core tube. The initial pretension loads of cable bolts were 50 kN (4.9 t) and 90 kN (8.8 t) respectively. Shear Tests were carried out using a 50 MPa concrete blocks. The shear displacement for the first test was limited to 50 mm, with the vertical shear load reaching 900 kN, the pretension load increasing to 238 kN and with no cable bolt failure. The cable failed in the second test at 60 mm shear displacement. The cable shear failure load was 1354 kN with the cable axial load reaching 385kN. It was observed that the majority of strand wires failed in tension and there were some in the combination of shear and tensile failures. Aziz, et al., (2015a) investigated the performance of 19 wire 21.8 mm diameter of plain and spirally profiled Hilti cable bolts subjected to double shear tests. The result for the plain cable bolt was higher than the spiral cable bolt because of reduction in the strength of spiral cable. The result of the double shear test was also compared with the single shear test recommended by British Standard (BS 7661-2- BSI 2009). It was found that the single shear test underestimated the shear strength of 21.7 mm, 19 wire (9 x9x1) cable bolt Rasekh et al., (2015) studied the contact surface area of the concrete joints during shearing. It was observed that the contact surface area can vary between 70-90% of the total surface area. Aziz et al., (2015b) proposed a mathematical model to determine the peak shear load of the pre-tensioned fully grouted cable bolts subjected to double shear test using the combination of Mohr-Coulomb Criterion and Fourier Series scheme. This model was extended by Aziz et al., (2016) to determine the peak shear load when concrete blocks were not in contact with each other. Rasekh et al., (2016a) compared the experimental test results with the mathematical models to determine their accuracy in simulating experimental studies. Rasekh et al., (2016b) used the Energy Balance theory and Fourier series concept to simulate the shear performance of cable bolts subjected to double shear tests in elastic,

strain softening and failure stages. Further study on the shear behaviour of various cable bolts under double shear tests without joints in contact with each other is the subject of this paper.

SAMPLE PREPARATION, TEST APPARATUS AND EXPERIMENTAL PLAN

Each double shear test employed three concrete blocks, comprising two 300 mm cubic blocks and a central rectangular block 450 mm long and 300 x 300 mm side. The casting of concrete blocks for the test was carried out in the steel frame of the double shear apparatus. Four wooden plates were used to stabilise concrete blocks and separate them from each other during casting. The assembled moulds were held together with appropriate clamps. A steel conduit wrapped by 3 mm diameter electrical wires was centrally placed through the mould to produce a rifled hole in the concrete as shown in Figure 1. The diameter of the steel conduit varied depending on the type of the cable tested. All wood plates, concrete blocks and the steel conduit were greased using petroleum jelly to prevent fresh concrete from sticking to steel plates, central conduit and wooden partitions. During concrete blocks casting, four 20 mm diameter plastic conduits were placed vertically along the central line of the mould to produce hollow vertical holes for grouting purpose.



Figure 1: Concrete blocks casting assembly

A modified double shear apparatus (MKIII) was used to remove the friction between concrete block joint surfaces during shearing process and to determine the pure shear strength of cable bolts. Two open box steel channel braces (U brace), mounted axially on each side of the double shear apparatus assembly and connected to two end plates with dimension of 500 x 340 x 30 mm³ was employed to prevent the concrete joint faces of the concrete blocks coming in contact with each other. During shearing the load was transferred from the U sections to the end plates thus avoiding the friction between shearing blocks (Figure 2). The 500 t machine in the laboratory of University of Wollongong was used to perform double shear tests. The rate of shear displacement was set by the digital controller at 1 mm/min. The shear load was applied to the sample using a hydraulic jack located on top of the instrument. The middle concrete block was moved in the vertical direction. The amount of shear and normal load and shear displacement were recorded by a data taker.



Figure 2: Double shear assembly without friction between concrete blocks

Thirteen double shear tests were conducted using five different types of cable bolts as shown in Table 1, showing the number of wire strands, cable bolt diameter, typical strand yield strength, lay length and elongation at strand failure. 40 MPa concrete blocks were used as host medium.

Cable bolt type	Wire Strand No.	Dia. (mm)	Typical Strand Yield Strength (kN)	Typical strand breaking load (kN)	Lay length (mm)	Elongation at strand failure (%)
Plain SUMO	9	28	568	635	400	5-7
ID-SUMO	9	28	568	635	400	5-7
Plain 19 wire(9x9x1)*	19	21.7	-	590	300	-
Plain MW10	10	31	-	687	600	5-6
Spiral MW9	9	31	-	608	600	5-6

Table 1: Cable bolt properties

* 19 wire (9x9x1) twin layer seal construction cable strand

TEST RESULTS AND DISCUSSION

Table 2 shows the result of conducted tests including peak shear load and the corresponding shear displacement. This provides a good comparison with regard to cable bolt type (surface profile type, lay length and ultimate tensile strength) and pretension load. Plain SUMO with 0 t pretension load, Plain MW10 with 0 t pretension load and Spiral MW9 with 0 t pretension load did not reach their peak shear load in 100 mm of shear displacement. This table shows the percentages of shear strength of each cable bolt to its Ultimate Tensile Strength (UTS). The average is about 67%, which shows that the shear strength of a cable bolt is approximately 65% of its UTS value as expected based also on the theoretical calculation

Test No.	Cable	Cable Dia. (mm)	Hole Dia. (mm)	Pre- Tension (kN)	Peak Shear Load (kN)	Shear Displacement at Peak Shear Load	Shear Strength/ UTS (%)
		()	()	(1.1.1)	(111)	(mm)	0.0(/0)
1	Plain 19 wire (9x9x1)	21.7	30	0	884	76.8	74.9
2	Plain 19 wire (9x9x1)	21.7	30	0	761	98.8	64.5
3	Plain 19 wire (9x9x1)	21.7	30	100	738	92.3	62.5
4	Plain 19 wire (9x9x1)	21.7	30	160	774	86	65.6
5	Plain SUMO	28	42	0	886*	100	69.8
6	Plain SUMO	28	42	150	852	88.2	67.1
7	ID-SUMO	28	42	0	815	93.4	64.2
8	ID- SUMO	28	42	150	767	85.7	60.4
9	Plain MW10	31	42	0	878*	105	63.9
10	Plain MW10	31	42	150	923	88.5	67.2
11	Spiral MW9	31	42	0	939*	105	77.2
12	Spiral MW9	31	42	75	907	89.7	74.6
13	Spiral MW9	31	42	150	837	88.5	68.8

Table 2: Test results

* The shown value is not peak shear load because the cable did not break. Stratabinder grout was used to install the cables in 40 MPa concrete.

Effects of pretension load

Figure 3 shows the effect of pretension load on the peak shear load of various cable bolts. It was observed that the peak shear load decreases by increasing the pretension load of all the five types of tested cable bolts. The exception to the trend was the peak shear load for Plain MW10 as the cable bolt did not reach its peak shear load at 0 t pretension load. Moreover, the shear displacement at peak shear load decreased by increasing the pretension load. Further tests are planned for MW10 cable bolt. Also note that the four plain 19 wire cable bolt results are arranged in two groups (tests 1 and 4 in one group and tests 2 and 3 in another). This is because of different techniques were used to stop grout leaking out of the blocks during cable bolt installtion process.



Figure 3: Effect of pretension load on peak shear load

Effect of cable type

Figures 4 and 5 show the comparison between peak shear loads of different types of cable bolts subjected to 0 and 15 t pretension loads. The result shows that Plain MW10 had the highest peak shear load at 15 t pretension loads. The comparison in 0 t pretension load was difficult with other tested cables because the plain SUMO, spiral MW9 and plain MW10 did not fail in 100 mm of shear displacement. The higher shear strength of plain MW10 was due to the cable strand having ten wires instead of nine wires of SUMO and MW9 cable bolts. Other features of MW10 cable include high tensile strength of 70 t, greater wire lay length of 600 mm and plain cable surface profile type.

Cable bolt's mode of failure

Figures 6 and 7 show the failure angle and modes of failure in MW10 cable bolt. The cable bolt failed completely on one side. As Figure 7 shows, the failure in the cable bolt was observed to be a combination of both shear and tensile failure.

Cable bolt's deflection

Figure 8 shows the hinge point deflection in the cable bolt. The level of deflection at the hinge point is clearly evident from the extent of deformation shown in Figure 8. Result from Table 3 illustrates that the deflection in tests were between 65 mm and 110 mm with an average of 85 mm. The amount of cable deflection was more than three times of its diameter.

Equipment modification

It is clear from this study that, due to the addition of braces on the double shear testing apparatus; the results of cables in shearing were found to be more consistent with past studies where the influence of joint surfaces occurred. However, the concrete deformation at hinge points is excessive, which leads to the cracking of concrete blocks. One way of achieving comparative shear failure values of cables, in both the single and double shear tests, would require shearing the cables in concrete blocks of similar confinement. Figure 9 shows the new version of the double shear apparatus which will allow equal confinement to be applied to the host medium. Such confinement will allow identical torqueing of the concrete medium irrespective of the testing technique. Thus the applied confident loads will be similar to tests conducted in single shearing as well in cable pull tests carried out by Chen *et al*, (2016), and Mackenzie *et al* (2014) and Aziz, *et al*, (2017).







Figure 5: Peak shear loads of different types of cable bolts subjected to 15 t pretension load



Figure 6: Failure angle of cable bolt



Figure 7: Tensile and shear failure of cable bolt



Figure 8: Cable bolt deflection



Figure 9: Double shear apparatus MKIV

Test No.	Cable	Pre-tension (kN)	Dia. (mm)	Cable bolt deflection at hinge point mm)	(Cable bolt deflection) Diameter
1	Plain 19 wire (9x9x1)	0	21.8	65	2.98
2	Plain 19 wire (9x9x1)	0	21.8	70	3.21
3	Plain 19 wire (9x9x1)	100	21.8	60	2.75
4	Plain 19 wire (9x9x1)	160	21.8	85	3.90
5	Plain SUMO	0	28	90	3.21
6	Plain SUMO	150	28	110	3.93
7	ID-SUMO	0	28	95	3.39
8	ID-SUMO	150	28	75	2.68
9	Plain MW10	0	31	100	3.23
10	Plain MW10	150	31	105	3.38
11	Spiral MW9	0	31	90	2.90
12	Spiral MW9	75	31	90	2.90
13	Spiral MW9	150	31	105	3.39

Table 3: Comparison between cable bolts deflection and diameter

CONCLUSION

The shear performance of pre-tensioned fully grouted cable bolts were studied by conducting a series of experimental double shear tests on five different types of cable bolts without contacts of concrete blocks surfaces to determine their pure shear strength. The following conclusions are drawn from this investigation:

- The plain cable bolt provides higher shear strength irrespective of the cable type.
- The shear load and shear displacement at peak shear load decreases by increasing the pretension load.
- The shear strength of each cable bolt was 67% on average of its Ultimate Tensile Strength.
- The double shear tests should be modified to permit the application of confinement load to the host medium. The availability and consistency of the concrete confinement would enable comparative tests to be carried out on tendons irrespective of the shear apparatus types as long as other factors and parameters remain the same.

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THE EXTENT OF SHEARING AND THE INTEGRITY **OF PROTECTIVE SLEEVE COATING OF CABLE** BOLTS

Naj Aziz¹, Ali Mirzaghorbanali² and Matthew Holden³

ABSTRACT: Long term integrity of cables installed around tunnels for ground reinforcement can be influenced by ground movement. This paper reports on the laboratory study of the influence of shearing on damaging the encapsulated plastic sleeves leading to exposure of the cable surface to a hostile environment. Two experimental studies were carried out to assess the extent of shearing displacement and damage to the sleeves. Various shear displacement tests were carried out. In the first test a sleeved cable was encapsulated in a plastic tube and single sheared up to 43 mm vertical displacement and the same procedure was repeated with the second sleeved cable being subjected to double shearing using a double shear testing Machine MKII. In both tests it was found that the corrugated plastic sleeves started to be sheared at a maximum displacement of greater than 20 mm, without damage, it was inferred that the corrugated sleeve can withstand shearing displacement without tearing up to maximum of 33 mm. The experimental procedure and the variation in the testing method are described.

INTRODUCTION

Increasingly tunnels are being introduced into metropolitan transport systems to provide links in urban areas where surface routes become congested and are preferred to conserve surface facilities of particular merit. Tunnels provide safe, environmentally sound, very fast and unobtrusive transport for all walks of life. In urban areas they are also built or constructed at shallow depth and pass through different rock structures of varying competence. Construction of tunnels in urban areas requires effective reinforcement and regular monitoring. Failure to address reinforcement integrity may have severe consequences including;

- Damage to the tunnel structure caused by excessive tendon corrosion, •
- Interruption to traffic flow, •
- Excessive tunnel maintenance cost, ٠
- Damage to surface facilities, and •
- Costly litigations •

The most widely used reinforcement system now-a-days is by tendons (both rock bolts and cables) and their effectiveness and long term performance is dependent on the nature of ground formation that the tunnel is driven through. Long term stability of the tunnel requires long term integrity of the reinforcement elements anchored into the surrounding rock formation. Steel corrosion represents the most important factor that undermines the long term integrity and stability of the constructed tunnels. The incorporation of plastic sleeves to tendons provides this element of protection as long as cables are encapsulated in the plastic sleeve and that the used sleeves have long term durability. Ground movement and deformation surrounding the tunnel may cause the sleeves to crack and exposure of the tendons to groundwater. The extent of the ground movement and tendon shearing may be

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evaluated by regular monitoring for ground movement from the surface. Better understanding of the extent of ground deformation that may contribute to the demise in the integrity of the installed sleeves represents a challenge that is being addressed and is the subject of study in this paper.

A challenge associated with incorporating a protective plastic sleeve over the reinforcing tendon is ensuring effective load transference between the tendon, the grout annuli and the ground. A smooth plastic sleeve relies heavily on the skin friction between the sleeve and the grout annuli to transfer load from ground movement to the tendon and ultimately reduces the load transference capacity of the system. To overcome this, many ground support standards and guidelines specify that the geometric profile of the sleeve should be corrugated and sinusoidal in shape. For instance, the British standard code of practice for ground anchorages, BS 8081: 1989, as well as the Roads and Maritime Services (RMS) quality assurance specification for soil nailing (R64) both specify sinusoidal corrugations with a pitch between six and twelve times the sleeve wall thickness and amplitude not less than three times the wall thickness as shown in Figure 1. The idea is to create a mechanical interlock between the inner and outer grout annuli through the geometric interference introduced by the corrugations in the plastic sleeve. This relies on the shear strength of the cement grout as opposed to the mechanical properties of the plastic sleeve. Figure 2 shows a typical flexible corrosion protected ground anchor which incorporates the protective plastic sleeve.



Figure1: Cross-section of corrugated plastic sleeve



Figure 2: Typical flexible corrosion protected ground anchor

THE PROCEDURE

For evaluating the integrity of encapsulating sleeves on cable bolts, two methods of testing the sleeved cable sections were undertaken. The aim was to determine the maximum shear displacement of the cable that would cause the plastic sleeve to crack during shearing. Initially sleeved cable sections were subjected to a single shear test method and this was followed by the standard large scale double shear test methods. 21.7 mm diameter 19 wire (9x9x1) construction designation Superstrand cable was used in the study. BluGeo HS400 grout was used to encapsulate the steel

cable with corrugated plastic sheathing. The corrugated plastic sheath is manufactured from High Density Polyethylene (HDPE) with a wall thickness of 2.0 mm, pitch of 22 mm and amplitude of 6 mm (refer to Figure 1). A close up view of the cable and corrugated plastic sleeve is shown in Figure 3.



Figure 3: A close up view of the 21.7 mm diameter steel cable and corrugated plastic sleeve

Single shear test

A guillotine type single shear apparatus as shown in Figure 4a was used to carry out the preliminary shear tests. The encapsulated cable was grouted in a 5 mm thick smooth wall plastic tube using a cementitious grout to act as the outside protection layer. A 12 mm ring strip of the plastic cover was removed from mid-section of the encapsulated cable section to expose the corrugated tube, shown in Figure 4b, to allow the bare corrugated plastic sleeve to be visually inspected when sheared. Shearing of the cable was carried out in four displacement steps, until cracks appear in the corrugated sleeve.



Figure 4a: Single shear apparatus:



Figure 4b: Cable section with corrugated sleeve installed in 45 mm plastic tube

Figure 5 shows the shear load and shear displacement of four tests. The final test was terminated at shear displacement of 43 mm. The test was stopped at the end of each predetermined displacement step shown in the graph and the cable sleeve was physically examined for any damage. Four shear displacement step ranges were made. They were 6, 12, 24, and 40 mm ranges. As seen in Figure 5 the displacements range step of 24 mm represented, the critical shear travel for plastic sleeve failure. The shear displacement of the cable after 6 and 12 mm was recovered once the sheared load was taken off the cable. Figure 6 shows the ultimate cable shear travel and the condition of the damaged sleeve respectively. The final view of the damaged sleeve may have occurred, when the shear displacement was beyond 24 mm. An audible cracking sound was heard at the vertical displacement of around 33 mm. Therefore it is reasonable to suggest that cracking of the corrugated sleeve occurred at the vertical shearing movement of 33 mm.



Figure 5: Shear load and displacement graphs



Figure 6: Damaged corrugated sleeve at the end of the shearing test

Double shearing method

The aim of this investigation was to determine the possible damage on the sheath of the corrosion protected cable bolt upon subject to 15 and 20 mm shear displacements. Testing was carried out in accordance with the double shearing methodology reported by Aziz *et al.*, (2015 *a* and *b*). In this study the contact between concrete medium joint surfaces were allowed, by using MKII double shear apparatus. The double shear testing process requires three concrete blocks with two outer 300 mm side cubes and a central rectangular block 450 mm long. The strength of the concrete used was relatively weak at around 20 MPa as specified for the investigation. The casting of the concrete blocks was carried out directly in the confining steel frame of the double shear apparatus. A plastic conduit
wrapped with 8 mm PVC tube and set through the centre of the mould lengthways, creating a centralised hole for cable installation in the concrete blocks. The plastic conduit was gently pushed out once the concrete block was set. The concrete blocks were left immersed in a water tank to cure for a minimum period of 28 days. The Uniaxial Compressive Strength of concrete was determined as 21 MPa, after the period of curing, by testing three cylindrical samples.

The cured blocks were then mounted in the double shear confining steel frames and the sleeved cable bolt specimen was inserted into the borehole. The annulus section between the sheath and cable was grouted and left for setting prior to cable bolt installation. Two 100 t load cells were inserted onto each end of the cable followed by the typical cable bolt end fitting. The load cells were connected to the data logger during tensioning. Once the cable was pretensioned for 5 t of axial load as specified, the grout was injected into the annulus between the cable and borehole through the intersecting small holes on top of the block. The whole assembly was then left undisturbed for the duration of seven days for the grout to cure. 50 mm cube samples were cast from the same grout as used for encapsulation and then tested for strength, yielding 45 MPa of UCS after seven days of curing. The top of the concrete blocks were covered by the bolted steel plates and the whole assembly was then mounted on the carried base platform. The whole double shear assembly and the base frame was then positioned on to the 500 t compression testing machine for shearing process at the rate of 1 mm/min as shown in Figure 7.

RESULTS AND ANALYSIS

Figure 8 shows the shear load and axial load profiles against shear displacement for two tests conducted in this study. The maximum values of shear load attained during double shearing, were 20 t and 24.3 t for 15 mm and 20 mm of shear displacement respectively. These correspond to maximum axial load of 7.3 t and 8.9 t. At the end of the test, the double shear assembly was dismantled and the tested cable was extracted and examined for the extent of damage to the protective sheath. Figure 8 shows pictures that were captured once the concrete blocks were gently dismantled.



Figure 7: Double shear testing apparatus in 500 t compression testing machine



Figure 8: Shear and axial load profiles against shear displacement, left at 15 mm of shear displacement and right at 20 mm of shear displacement

For 15 mm of shear displacement;

- No damage was observed cable bolt sheath,
- Deformation shear displacement of the cable bolt was recovered once the sample was dismantled,
- Cracks were observed on top of the concrete blocks once the steel cap was opened.

Figure 9 shows post-test picture of the second test where the maximum shear displacement set to 20 mm. The following main conclusions were obtained:

- No damage on sheath of cable bolt was observed,
- Permanent deformation of the sheathed cable was noted once the sample was dismantled,
- Cracks were observed on the top and side of the concrete blocks once the steel box was opened.

Clearly the vertical shear displacement of 20 mm appeared not to have caused any detrimental damage to the sheath, which entails that the enclosed cable inside the plastic sheath would not be exposed to an adverse environment. Triggering of the plastic deformation and cracking would be likely to occur at shear displacement beyond 25 mm as indicated from the initial single shear testing.



Figure 9: Post-test pictures of double shearing for 20 mm of shear displacement

CONCLUSION

No damage was noted on the protective sheath of the cable bolt when the double shearing assembly was subjected to 15 and 20 mm of shear displacement at the rate of 1 mm/min. Permanent deformation was observed on the protective sheath after 20 mm of shear displacement. The position of permanent deformation corresponds with the location of the concrete blocks joint. Triggering of the plastic deformation and cracking would likely to occur at shear displacement beyond 25 mm as indicated from the initial single shear testing.

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A NEW INDIRECT METHOD FOR EVALUATION OF THE SWELLING POTENTIAL OF ARGILLACEOUS ROCKS

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ABSTRACT: One of the most important characteristics of argillaceous rocks is their swelling potential. This can be determined either by direct or indirect methods. Conventional tests are performed in which one of the properties/indices of rock is associated with the swelling potential. Since the direct tests are usually costly and time consuming, this article is focused on introducing a new method for evaluation of the swelling potential of argillaceous rocks. This is a quicker and less expensive test. The present study shows that there is a good correlation between "Contact Angle" of a drop of water on a flat rock surface with its swelling potential. Since the swelling potential is associated with molecular structure and surface tension of molecules, it is expected that contact angle (which is also influenced by this surface tension) be correlated with swelling potential.

In the present study, free swelling, contact angle, plastic and liquid limit tests are conducted on rock samples and their correlations are determined. The results showed that although there is an exponential relationship between plasticity index and swelling strain, using this parameter as an indirect method for swelling evaluation, has some limitations. On the other hand, due to the ability of contact angle to distinguish rocks with different swelling potentials, using this parameter has been proven to be an appropriate criterion for assessment of swelling potential.

INTRODUCTION

The swelling phenomenon, according to the definition of ISRM (1983), is a combination of physicochemical reactions in rocks with involvement of water and stress relief. Swelling occurs in soils or rocks with clay, anhydrite or pyrite/marcasite minerals, Barla (2008).

Swelling of rocks causes major problems for rock engineering projects both during construction and over in the operational life. In Belchen tunnel in Switzerland, marl, anhydrite and opalinus clay were excavated which presented problems like heaving of invert and cracking of drainage pipes soon after excavation. A 17 m high cavity was formed at the roof of the Sallsjo tailrace tunnel in Sweden about a year after commissioning of the tunnel. This was due to the presence of a 3 m wide shear zone containing montmorilonite clay. Singh and Goel (2006) reported that over 31 years of service in Bozberg tunnel in Switzerland, invert heaves of 27 and 33 cm were observed in anhydrite and opalinus shales respectively. Similarly, due to increased swelling pressure behind the concrete lining of Masjed–Soleiman underground power house cavern (PHC) in Iran, cracks were generated in the concrete lining at the contacts with mudrock layers as reported by Doostmohammadi (2008). Due to problems of swelling rocks that can cause, the evaluation of this parameter has a significant role in engineering projects, which cross such formations.

Evaluation of the swelling potential of rocks can be done either by direct or indirect methods. Conventional tests such as free swelling described by ISRM (1983) are classified as direct methods in which the rock sample is exposed to water and the resulting swell is measured directly in the surface of rock. Most of the direct methods for swelling tests are time consuming and costly. Such laboratory tests may take from a few days to even a few years as performed at Karlsruhe University, Mutschler (2003). However for indirect methods, which are usually very fast relationships are provided, which

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correlate this potential to one of the properties/indexes of the rock. The duration of long test for swelling measurement is usually not acceptable by many projects especially at the early stages of their feasibility study. In these circumstances, a quick and accurate enough test can be very useful for swell evaluation.

Various indirect methods for assessing swelling capacity of expansive soils have been proposed so far as explained by Asgari and Fakher (1994). Indirect methods use one or more soil parameters (such as percentage of clay–size fraction (< 2μ m), activity, density, plasticity index, liquid limit or water content of soil specimen) and relate it to swelling potential.

This paper aims to establish a new indirect method for evaluation of swelling potential in rocks. This is based on the concept of surface tension of a rock sample and its wettability property. Wettability is defined as the preference of a solid to attract a liquid or gas (known as the wetting phase) rather than another as described by Vijapurapu (2002). In other words, wettability is defined as the ability of one fluid to spread or adhere to a rock surface in the presence of other immiscible fluids. The "contact angle" test is a quick test which can be utilized for this purpose. To make a comparison between the correlation of this new method and older ones foo the swelling potential, a series of plasticity index tests were also performed.

SWELLING MECHANISM

Swelling is caused by one or a combination of three mechanisms as explained by Einstein (1996): mechanical, osmotic and intra-crystalline. Mechanical swelling is caused by dissipation of excess pore pressure. Osmotic swelling is related to the double layer effect, i.e. the large difference in concentration between ions held electrostatically close to the clay particle surfaces and the ions in the pore water further away. Osmotic swelling is controlled by the interaction of repulsive forces related to the double layer effect and externally applied stresses. Klein (2001) described the intra-crystalline swelling which is caused by hydration of the exchangeable cations. The cations hydrate upon contact with water and arrange themselves in a plane halfway between the clay layers. This results in a widening of the space between the clay layers and the overall effect depends on the number of water layers between the clay layers.

Occurrence of each mechanism depends on the rock forming minerals. For example, osmotic swelling occurs in clay bearing rocks while interacrystalline swelling occurs in smectite and mixed layer clays in anhydrite and in pyrite and marcasite.

The main groups of crystalline materials that make up clays are kaolinite, illite and montmorillonite. These are often called swelling or expansive clays. One feature of these minerals is having high specific surface (surface area per unit mass). The specific surface of kaolinite ranges from 10 to 20 m²/gr while for illite it ranges from 65 to 100 m² per gr. Montmorilonite can have a specific surface as high as 1000 m² /gr. Large surface area in this group cause significant role of surface forces in the behavior of clays.

The surface charges on clay minerals are negative (anions). These negative surface charges attract cations and positively charged side of water molecules from surrounding water source. Consequently, a thin film or layer of water, called absorbed water, is bonded to the mineral surface. The thin film or layer of water is known as diffuse double layer (Figure 1). The largest concentration of cations occurs at the mineral surface and decreases exponentially with distance away from the surface.



Figure 1: Diffuse double layer

The swelling of purely argillaceous rocks (i.e. rocks not containing anhydrite) is for all practical purposes sufficiently well understood. Under tunneling conditions, the swelling of argillaceous rocks is of an osmotic nature. This means that the cation concentration is lower in the pore water than close to the surface of the clay particles. This is because the latter (due to their negative electrostatic charge) attracts the cations (Figure 2). To compensate for the difference in concentration, pore water enters into the space between the clay particles (osmosis) and forces them apart. This can be prevented by applying a counter pressure, which theoretically decreases rapidly nonlinearly with the distance between the clay particles.



Figure 2: Double layers around clay particles

Gouy double layer theory is a theoretical method for calculation of swelling pressure using mineralogical parameters. Some of the parameters considered in this method are specific surface area of clay fraction, cation – exchange capacity (CEC), ion concentration that is far from the clay surface and electrical potential that is midway between the clay surface, Keith (2008). Therefore, according to this method, mineralogical investigation is required to assess the swelling pressure.

PLASTICITY INDEX AND SWELLING

For a long time, swelling was known to be a close function of the plasticity index. The plasticity index is the range of water content where the soil exhibits plastic behavior, Budhu (2008). This parameter is the difference between the liquid limit and the plastic limit of a soil. Both liquid and plastic limits have been developed by Attherberg, a Swedish scientist, to describe the consistency of fine-grained soils with varying moisture content. The moisture content at the point of transition from semisolid to a plastic state is the plastic limit, and from plastic to liquid state is the liquid limit. The plastic limit is

determined by rolling a thread of a fine portion of a soil sample on a flat non-porous surface, while the Casagrande cup is used to determine the liquid limit.

Since both swelling potential and plasticity index are functions of the amount of water absorbed by clay, plasticity index has widely been used for indirect evaluation of swelling potential in expansive soils as well as in weak rocks. Figure 3 shows a sample of such relationships.



Figure 3: Relationship between free swell and plasticity index for various rocks (Klein, 2001)

WETTABILITY AND SWELLING

Considering the important role of clay minerals and their surface properties both in swelling mechanism and wettability, it was decided to use the correlation of these two as a method for evaluating swelling potential. Many different methods have been proposed for measuring the wettability of a system. Contact angle is one of the quantitative methods for measuring wettability. Contact angle describes the shape of a liquid drop on a solid surface. The shape of the drop reveals information about the chemical bonding nature of the surface. This bonding determines its wettability and adhesion, Vijapurapu (2002).

Chemical bonds are the attractive forces between atoms in a molecule and between adjacent molecules in a substance. These are the forces that hold things together. When molecules are in close proximity with a liquid or solid, the atoms arrange themselves to optimally satisfy the bonding forces with nearby neighbors. Consider the idealized solid as shown in Figure 4.



Figure 4: Schematic of an idealized solid surface (Vijapurapu, 2002)

An atom in the interior has satisfied bonds in all directions: four in this 2-D drawing and six in the real 3-D world. But the atoms in the top row do not have one bond satisfied, because there is no neighbor above. These unsatisfied bonds constitute surface energy; a potential energy in the sense that another object brought up close might satisfy some of these "dangling" bonds. These bonds are the sources of wetting and adhesion. Hence, the contact angle is used to estimate the nature and strength of these bonds.

As mentioned before, swelling potential is dependent on the type of the clay minerals and the related parameters of the clay's surface that can be associated with the nature and the chemical bond of the surface. Therefore, the idea of using contact angle for developing a new indirect swelling evaluation method was established based on the relationship between contact angle and the nature and strength of the surface chemical bond. On the other hand, ease of the sample preparation, visible results and the high speed of the contact angle measurement test are other reasons for the choice of contact angle.

EXPERIMENTAL STUDIES

For the present laboratory tests, marl and mudstone samples were obtained from a water diversion tunnel of the Nargesi Dam site located in the Mishan Formation and the Bakhtiari Formation in Iran.

In order to identify the relationship between swelling strain and contact angle, both free swelling and contact angle tests were conducted on five types of samples. These five types included three types of marls (A, B and C) and two types of mudstones (E and F). For the free swelling tests, the method suggested by ISRM (1983) has been used and for the contact angle test, the device shown in Figure 5 was utilized. As mentioned before, the contact angle is the angle between a tangent drawn on the drop's surface at the resting point and a tangent to the supporting surface. Using a computerized image processing program, the contact angle is measured from a photo taken with a high resolution camera of the drop at the time of it contacting the resting surface.



Figure 5: Contact angle apparatus (Institute of Petroleum Engineering, University of Tehran, Iran)

In addition to the contact angle tests, plastic limit and liquid limit tests were performed (according to ASTM under designation D-4318) to determine the relationship between plasticity index and swelling potential.

TEST RESULTS AND DISCUSSION

The results of pictures taken from the shape of the liquid drop on the rock samples surface and their contact angles are shown in Figure 6. The results of contact angle and their associated free swelling

tests are summarized in Table 1. According to this, it can be concluded that surface energy of mudstone samples is more than the marl samples because the contact angle of mudstone samples is less than for the marl samples.

		J
Sample	Contact angle (degree)	Maximum Swelling Strain (%)
Α	21.32	1.86
В	27.80	0.8
С	30.08	0.02
D	10.63	16
Е	17.96	6

Table 1: Results of free swelling and contact angle tests



Figure 6: Shape of a liquid drop on a rock samples surface and them contact angles

The obtained results are drawn in Figure 7 which shows that swelling strain decreases with increasing contact angle. The dependency of these two parameters together is very strong (correlation coefficient 99%) represented by the following equation.

$$\varepsilon_{\rm r} = -0.0002a^3 + 0.0631a^2 - 3.1277a + 42.4 \tag{1}$$

Where \mathcal{E}_s is the swelling strain and the contact angle.



Figure 7: Relationship between swelling strain and contact angle

(2)

Plastic and liquid limits of rock samples obtained from related tests are given in Table 2. All samples based on Burmister qualitative classification of plasticity index (1949) are classified in medium plasticity category, Das (2002). The relationship between plasticity index and swelling strain is shown in Figure 8. This dependency is in the shape of the following exponential function in which swelling strain increases with increasing plasticity index.

$\varepsilon_s = 0.0223 e^{0.3477 PI}$

Sample	Liquid limit	Plastic Limit	Plasticity index
Α	32	19	13
В	30	20	10
С	34	29	15
D	38	20	17
E	37	20	17
18			
16			•
_ 14			
<u>گ</u> 12			1
10			
8 S			
velli 9			
5 4			
2			1 0
0		Samp	ole - C
8	10	12 14 2	16 18 20
		Plasticity Index (%)	

Table 2: Test results of liquid and plastic limits

Figure 8: Exponential relationship between swelling strain and contact angle

In the above equation, \mathcal{E}_s is the swelling strain and PI is the plasticity index. As shown in Figure 8, sample C does not follow the general trend of the curve. This means that there are some clays that behave plastically but do not show any swelling behavior. In fact all of the clays behave plastically (with different extents) but only some of them swell. Therefore plastic index is not a good criterion to be used for evaluating swelling behavior. On the other hand, contact angle is very sensitive to the clay minerals therefore sample C in Figure 8 is very well in line with the general trend of the swelling graph.

CONCLUSION

This paper has described a new approach to indirectly evaluate the swelling potential of rocks. Contact angle, free swelling, plastic and liquid limit tests were performed and relationships were proposed to show the dependency of swelling strain to the plasticity index and contact angle.

The results show that there is a nonlinear inverse relationship between swelling strain and contact angle. This relationship is described by a third order algebraic function (equation 1). On the other hand, an exponential relationship between swelling strain and plasticity index has been observed.

According to the inability of the plasticity index to make a distinction between clay mineral types, using the plasticity index as an indirect method for evaluating rock swelling is under question.

Unlike plasticity index, contact angle was proved to be an appropriate parameter for indirect evaluation of swelling potential of rocks. This is due to the fact that the contact angle is very well dependent on the mineral type and is influenced by their surface energy, the parameter which also is influential in swelling behavior. The conclusion was derived from the fact that the contact angle can distinguish between rocks with different swelling potential, because there is a relationship between the nature and strength of rock surface chemical bond and contact angle. Relatively easier sample preparation and faster tests and also visible results are other advantages of this test which makes it a superior swelling index measuring method in comparison with other swelling tests.

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THE EFFECT OF WATER SATURATION IN SANDSTONE AND LIMESTONE SAMPLES ON DISC CUTTING PERFORMANCE

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ABSTRACT: A study was undertaken to quantify the changes in disc cutter performance between dry and saturated rock. The tests were conducted using samples of limestone and sandstone to determine whether changes in performance are consistent across rock types. A linear rock cutting machine was used to conduct the tests with a rolling disc cutter. The tests were performed at varying depths with fixed disc spacing. Cutting forces were measured in the different rocks and cutting depths with rock yield determined and specific energy calculated.

Comparing the results between cutting in dry and saturated rock showed reductions in cutting forces of 52% and 7% respectively for sandstone and limestone. That is the magnitude of the reduction in forces when cutting saturated rock was not consistent with a significant difference between the two rock types. The changes in specific energy were similar with a 43% and 9% reduction in sandstone and limestone. This difference in behaviour between the two rocks was also reflected in rock strength. A comparison of the strength between dry and saturated rock found the change was non-uniform with a 64% and 17% reduction in uniaxial compressive strength in the saturated sandstone and limestone samples respectively. When using a rolling disc cutter, rock cutting performance and rock strength were found to alter between dry and saturated conditions. In the case of a sandstone sample there were significant reductions in cutting forces, specific energy and strength whereas in limestone there were only marginal reductions in these parameters.

INTRODUCTION

The growth in global population has brought many challenges, including mounting pressure on developing infrastructure as well as increased demands on mineral consumption. These relate to both developed as well as developing countries. As the population continues to grow the supporting infrastructure must keep abreast in maintaining a sustainable standard of living. Much of this growth will occur in urbanised areas where there are already limitations on surface land use consequently there is likely to be more subsurface infrastructure development. In recent decades there has been a commensurate increase in the amount of underground construction despite its high cost. The North West Rail Link in Sydney, Australia, is one example of how high human densities are forcing infrastructure underground with the development of Australia's largest public transport project consisting of twin 15 km railway tunnels (Transport for New South Wales, 2015).

Minerals and metals are the building blocks for our modern society. They are key to all services, infrastructure and technologies as we know today (International Council on Mining and Metals, 2012). As near-surface mineral resources are depleted the mining sector faces the major challenge of discovering and accessing deposits that are deeper underground. It is critical that new tools and methods be developed to enable the economic recovery of these deep deposits. Mining methods such as block caving are in use in many parts of the world, which allows the economic extraction of deep underground ore deposits. However, the costs and advance rates related to underground development are significantly under par (Albanese and McGagh, 2011). Actual mine data reveals that even though equipment technology in the mining industry has improved, development performance has not kept paced and is substantially less compared to those achieved in the civil industry. Underground development rates in the mining industry are typically 5 m/d whereas civil projects

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achieve on average 10 m/d. The implementation of proven civil tunnelling technologies such as Tunnel Boring Machines (TBM) and the reliable prediction of rock behaviour and cutter performance will improve the speed and quality of underground development in the mining industry. TBMs have been used in mining operations as early as the 1950s (Cigla, *et al*, 2001). The Grosvenor Mine owned by Anglo American Metallurgical Coal is one of the few examples that show the effectiveness of bringing together previous experience and knowledge from the civil tunnelling industry and implementing it in the mining industry with the development of two access drifts using a TBM (Donnelly, *et al*, 2014).

Machines such as raise borers and TBMs are used to excavate hard rock. These excavators usually employ disc cutters which have proven their effectiveness in hard rock conditions, both in the civil and mining industries. Disc cutting excavators are attractive for hard rock applications because they provide: higher advance rates; safer and more stable excavations; and, can create smooth tunnel profiles (Wilson and Graham, 1972). Information about the forces that act on a disc cutter and their performance is predominantly based on testing conducted in dry rock. While in some cases disc cutters are used in dry rock, more often development occurs in wet conditions in which rock strength properties and cutting performance can be markedly different hence it is important to assess disc cutting behaviour in these conditions (Mammen *et al*, 2009; Summersby, 2013; Abu Bakar, *et al*, 2014). Increased knowledge and quantification of how cutting performance changes in dry and saturated rock will improve the reliable estimation of production and development rates. In turn this will enhance the ability to develop accurate schedules and plans reducing potential financial project risk (Abu Bakar, 2012).

TEST SAMPLE PREPARATION

Rock properties

Testing was undertaken to assess the changes in uniaxial compressive strength and tensile strength of limestone and sandstone samples with moisture content. Testing was conducted to determine the key properties of four different rock conditions i.e. dry limestone, saturated limestone, dry sandstone and saturated sandstone. In total, 40 rock specimens were tested. The limestone and sandstone had average dry densities of 1.60 t/m³ and 2.22 t/m³ and saturated moisture contents of 25% and 4% respectively. Table 1 and Table 2 indicate the strength reduction while roughly consistent for each rock type, varied dramatically between the two rock types.

	Limestone (MPa)	Sandstone (MPa)
Dry specimen	5.3	67.4
Saturated specimen	4.4	24.1
Strength reduction	17%	64%

Table 1: Reduction in uniaxial compressive strength when saturated

Table 2: Reduction in tensile strength when saturated

	Limestone (MPa)	Sandstone(MPa)
Dry specimen	0.4	1.2
Saturated specimen	0.3	0.3
Strength reduction	14%	76%

Sample preparation

The preparation of the dry test block samples involved oven drying at 105° for seven days. This temperature and duration was to minimize if not eliminate moisture in the samples, the blocks were then stored in a cool dry place until they were tested.

Other sample blocks were saturated through a technique known as incremental saturation. This involved placing the blocks in a large plastic box. The box was then filled with water. The water level was increased incrementally for seven days until the block was completely submerged. This technique allows capillary forces to act, drawing water into the pore spaces in the rock while allowing air to escape. This incremental saturation method ensured that the center of the block was completely saturated. After seven days of incremental saturation, the blocks were submerged for eight weeks before being tested. Each of the sample blocks was encased in a medium hardness casting plaster. The plaster was mixed in small quantities in a steel bowl, to ensure an accurate measurement of the water-to-plaster ratio and then added to each block that had been placed in each mould. Foam sheets were used to cover the bottom and sides of the mould. The dry block samples were first placed in plastic to avoid moisture contamination from the plaster as shown in Figure 1. The plaster was allowed to cure for a minimum of 24 hours before being tested.



Figure 1: Dry block samples wrapped in plastic and ready for casting

EXPERIMENTAL PROCEDURES

The testing procedure followed for linear rock cutting tests was as follows:

- 1. After the plaster had cured for 24 hours, the excess plaster was scrapped to create a flat even surface on the bottom of the mould.
- 2. The mould was then placed on the table of the Liner Rock Cutting Machine (LRCM). A steel plate was placed in the block to measure the level of the block. This allowed the average level of the block to be measured. A magnetic digital protractor was used to record the average levels along the *x* and *y* axis. The mould was then fastened to the LRCM table with screws and tightened to level the block as much as possible as shown in Figure 2.
- 3. The distance between the disc and the block was adjusted precisely to ensure a consistent depth of penetration. This was achieved by measuring the distance offset between the disc and the block and setting the depth of penetration accordingly.
- 4. The block was then marked out to ensure that the first cut was 65 mm from the edge of the block in sandstone and 45 mm in limestone. Due to the hardness to the sandstone the distance between the edge and the cut was larger to prevent the sample from failing at lower depths of cut.
- 5. A foam containment structure was placed around the mould. This foam containment structure contained all chips and fine material that were ejected during a cut.
- 6. The cut was performed and data from the load cell acquired.

- 7. The ejected rock chips and fine material was collected using a vacuum collection device. This vacuum device was created to fit onto the end of a conventional vacuum cleaner. The device was able to achieve a material recovery of approximately 98 %.
- 8. The collected material was then weighed. The debris was then placed in the oven at 105° for 24 hours and weighed again to determine the moisture content of the cut.
- The LRCM table was then setup for the next cut at a spacing of 40 mm. Due to the distance left from the edge of the blocks, two adjacent cuts were performed per block in the sandstone and three in the limestone;
- 10. This process was then repeated for different penetration increments with several passes to investigate the effect of groove deepening. Figure 3 shows a schematic of this nomenclature.



Figure 2: Block sample fastened to LRCM table prior to cutting (left) and foam containment structure to retain cutting debris (right)



Figure 3: Schematic of the LRCM sample and nomenclature (Balci, 2013).

EXPERIMENTAL RESULTS

Observations

Figure 4 shows the force profiles observed from a cut in limestone and in sandstone. The primary difference between the two force profiles is the magnitude of the forces. The force in cutting the sandstone is nearly an order of magnitude much greater than the limestone. In addition, the force trace is also noticeably different. The limestone force trace is relatively uniform and flat whereas the sandstone trace shows distinct peaks and troughs. Due to the low strength of the limestone, the thrust force applied by the disc caused localised crushing (Figure 5). The peaks and troughs in the sandstone trace indicate the formation of chips. As the disc passes through the rock there is a build-up of elastic energy in the rock indicated by the peaks. This build of elastic energy is then suddenly

released resulting in a sudden drop in cutting forces. This mechanism is what causes chips to form in the sandstone



Figure 4: Typical force profile during cutting in limestone (left) and sandstone (right)



Figure 5: Localised crushing in limestone (left) compared to substantive rock chips in cutting sandstone (right)

RESULTS

Cutting is classified as either unrelieved or relieved. Unrelieved cutting in rock occurs in the absence of any previous cutting whereas relieved cutting refers to the situation that occurs when a previous cutting has been made that can lead to formation of microfractures that can assist subsequent cutting.

Three cuts were made across the surface per incremental penetration in the limestone samples while only two cuts were made in the sandstone sample. This was due to the different edge spacing used in the sandstone and limestone to prevent the samples from failing at low depths of cut. The first cut in the surface was made as unrelieved cut and subsequent cuts were relieved cuts. Since three cuts were made in the limestone, the average of the two relieved cuts was calculated. However, since only two cuts were made in the sandstone, there was only one data measurement for the relieved cut.

The effect of water saturation on thrust force in unrelieved cutting in sandstone and limestone is summarised in Table 3. The results show an increase in forces over the limited range of penetration but that the effect of water had a different effect in the two rock types. As shown in Figure 6 there was a significant reduction in thrust force that tended to increase with penetration when cutting in the saturated sandstone sample as compared to the dry sample. However, the reduction in thrust force in the saturated limestone was minor over the range of penetration as shown in Figure 7. Similar reductions were in rolling force in the saturated sandstone as shown in Figure 8.

Penetration	Limestone		Sandsto		ne	
(mm)	Dry (kN)	Saturated (kN)	Reduction	Dry (kN)	Saturated (kN)	Reduction
4	3.1	2.7	13%	14.9	7.6	49%
5	3.2	2.9	9%	16.6	9.2	45%
6	4.7	4.2	11%	26.1	10.8	59%
7	5.1	5.2	0%	29.7	13.7	54%

Table 3: Reduction in unrelieved thrust force in two rock types with water content





Figure 6: Variation in thrust force with depth in cutting sandstone



Figure 7: Variation in thrust force with depth in cutting limestone



Figure 8: Variation in rolling force with depth in cutting sandstone.

CONCLUSION

The study was conducted to better understand how cutting performance with a rolling disc cutter might change when cutting in dry and saturated rock. It involved a comprehensive program of linear rock cutting tests to evaluate the differences in performance between cutting dry and saturated rock samples. The test sample blocks of two rock types, a limestone and a sandstone were used to gauge whether there might be any differences in effect with rock type.

Data collected from the linear rock cutting tests was analysed to find differences in cutting performance in dry and saturated rock conditions. An average reduction of 52% and 8% was observed in thrust force in the saturated sandstone and limestone samples respectively. Similar levels of reduction were found in rolling force. The average reduction in specific energy was 43% and 9% in sandstone and limestone. These reductions in forces and specific energy correspond with the 64% and 17% reduction in UCS in the saturated sandstone and limestone samples respectively.

The yield produced in the limestone at low depths during cutting was observed to consist predominantly of fine material. This was caused by localised crushing under the disc due to the low UCS of the limestone. However, the chip size when formed in sandstone chips increases proportionally with the depth of cut.

Compared to the reduction in cutting performance in the saturated sandstone sample, the change in cutter performance in limestone was less substantial. The effect of water saturation varies directly with penetration, indicating greater levels of cutting performance can be achieved with a disc cutting machine such as a TBM. Performance changes appear to be less effected in limestone.

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INCORPORATING MINE SEISMICITY AND COAL BURST CONSIDERATIONS INTO A STRATA FAILURE MANAGEMENT PLAN FOR COAL MINE ROADWAYS

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ABSTRACT: For coal mines the term bump should refer to a seismic event that is generated at some distance from the excavation and a burst should refer to a sudden uncontrolled fall of ground. There may be additional seismic noise generated at the excavation boundary directly associated with the fall of ground. The seismic sources for bumps are most likely to be the immediate or delayed failure of thick rock units in the overburden. For Australian coal mines, the sudden collapse of ribs should be considered to be either a strain burst in the context of the hard rock mining knowledge base or a gravity–driven kinematic failure (slump). As the depth of cover increases there is a greater thickness of failed coal at the excavation boundary and hence more material is available to be dislodged as a strain burst if the installed ground support is inadequate. Depending on the orientation of the roadway with respect to small-scale faults it is possible for wedges of coal to be defined, and these may be dislodged by a seismic bump. The dimensions of such wedges may be in excess of the maximum practical tendon length.

INTRODUCTION

The 2016 Work Health and Safety (Mines and Petroleum Sites) Regulation requires all mines to manage the health and safety associated with mining-induced seismic activity. There is a specific requirement for the principal hazard management plan for strata failure to include considerations of induced seismic activity. It is well known that mining-induced seismicity is associated with rock bursts in metal mines. There was little appreciation of the seismic hazard in Australian underground coal mines until the double fatality at Austar coal mine in 2014 which has been identified as a pressure burst (NSW Mine Safety Investigation Unit, 2015). The investigation report provided general guidance as to where a pressure burst hazard may be encountered – depths greater than 300 m, the presence of structures such as faults and dykes, changes in joint orientation, and the presence of massive roof or floor strata – and then infers a number of specific actions that should be adopted by mine management. Many of these actions would substantially slow the mine development and may make mining uneconomic.

This paper reviews the knowledge of coal bursts in the context of rock bursts in metalliferous mines and kinematic failure of vertical rock walls. It seeks to provide more clarity into how specific hazards can be identified and in particular the advantages and limitations of rib bolts and rib mesh support to limit the impact of the sudden onset of potential falls of ground.

MINING SYSTEMS

Much of the recent knowledge on mine seismicity and rock bursts comes from the metalliferous mining sector with perhaps the key publications being Kaiser *et al* (1996) and Kaiser and Cai (2013). Whilst the physics will be the same in the two mining sectors, there are substantial differences between the mining systems used in hard rock and those used in underground coal mining. These differences may require changes in the way the mine seismicity and burst hazards in coal mines are understood and managed.

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Coal mines

In Australian coal mines the development roadways are excavated using continuous miners or occasionally road headers. Drill and blast is rarely used, and then typically only for excavation through igneous dykes. The roadway excavations are rectangular in shape and typically 5.0 m - 5.5 m wide and 2.5 m - 3.5 m high. To extract additional coal longwall mining systems are used. Longwall extraction is conducted in sub-horizontal seams with the extraction voids currently ranging from 160 m to 410 m wide. Successful longwalling requires the immediate roof to cave, although the delayed caving of thick units (thickly bedded coarse sandstones or conglomerates) while not ideal is acceptable in some circumstances. A key requirement for high-production Australian longwalls is the lack of faults with throws more than the seam thickness; there are often small-displacement faults within the longwall extraction panels. To access suitable longwall reserves, development roadways may need to traverse larger throw faults and also fault zones with negligible net displacement.

The overall geotechnical environment for coal mining is dominated by the transverse isotropy introduced by the laterally persistent bedding discontinuities in the sedimentary rock mass. Uniaxial compressive strengths can range between 10 MPa and 100 MPa; the coal itself can range in strength from 6 MPa to 25 MPa. In Australia the maximum depth of mining is currently in the order of 600 m. Depending on rock and coal strength, the onset of brittle failure can occur at depths as low as 100 m. Figure 1 extends the excavation behaviour matrix first introduced by Hoek *et al* (1995) to highlight how the orthogonal joints and laterally persistent bedding produce an equivalent to a highly fractured rock mass. Note that kinematically unstable blocks (shown with the open arrows in Figure 1) can be readily defined by non-vertical discontinuities, bedding, and the excavation boundary and these will be present at all stress levels. Because such blocks will relax into the excavation they will not be exposed to elevated stress failures. The smaller zones of overstressing in the coal seam are ultimately related to the different stress field present in the coal seam ahead of mining compared to that in the roof and floor stone (Seedsman, 2004).



Figure 1: An additional set of excavation behaviour models for coal mine roadways, highlighting the possibility of kinematically-acceptable blocks at all stress levels

In Australian coal mines the roof and ribs are secured with bolts and mesh, typically installed off the continuous miners within about 2 m to 3 m the face. Bolt lengths can vary between 1.5 m and 2.4 m for the roof and typically between 1.2 m and 1.5 m in the ribs. Typically there are 6 to 8 bolts per metre in the roof, and 2 to 3 bolts per metre in the ribs. Longer flexible strands are often installed in the roof albeit with a substantial time penalty; longer strands into the ribs are not used. Mesh is in the form of welded mesh panels. By contrast, in the USA ribs are rarely supported; for example Hoelle (2009) summarises bumps in Eastern Kentucky, and although the seam was 3 m to 4 m high, rib support was not used (Hoelle, 2016).

In Australian coal mine terminology (NSW Mine Safety Investigation Unit, 2015), bursts are referred to as either:

- Pressure bump, or a bounce: A pressure bump is a dynamic release of energy within the rock mass in a coal mine, often due to intact rock failure or failure/displacement along a geological structure, that generates an audible signal; ground vibration; and potential for displacement of existing loose or fractured material into mine openings.
- Pressure burst: A pressure burst is a pressure bump that actually causes consequent dynamic coal/rock failure in the vicinity of the mine opening, resulting in high velocity expulsion of this broken/failed material into the mine opening. The energy levels, and hence velocities involved here can cause significant damage to, or destruction of conventionally installed ground support elements such as bolts and mesh.

A limitation with these two definitions is that the pressure burst definition requires high velocity expulsion which would have to be inferred from the geometry of the muck pile after the event. The possibility of gravity collapse without ejection is not covered. In US mines the terminology is different: Hoelle (2009) uses the term bump to describe events where coal is dislodged, with major bumps being defined as when mining had to be stopped, a significant quantity of coal was displaced, equipment damaged, and ventilation disrupted. A minor bump consisted of noise, small quantities of coal displaced, ground bounce, but no major large coal moved or equipment damaged.

Metal mines

By contrast the mining systems used in hard rock mines utilise drill and blast excavation techniques in roadways with an arched roof. Rock strengths are higher as are the maximum mining depths. The nature of metalliferous deposits is such that mining is often conducted in close proximity to major faults. Ground support typically utilises split sets, bolts, and fibrecrete.

A general assumption adopted to the geotechnical environment for hard rock mines is that the rock mass can be considered to be isotropic with no particular discontinuity set being dominant. This is one of the key assumptions that underlies the use of the Generalised Hoek Brown strength criterion and the Geological Strength Index (Hoek and Brown, 1997), and one that also allows the subsequent use of plasticity in the various numerical codes. These options are not as useful in transversely isotropic rock masses such as coal measures as the transverse isotropy alters not only the rock mass strength but also the way by which the stresses are redirected around excavations.

Rock bursts are defined as sudden ejection of rock associated with seismicity. Kaiser and Cai (2013) distinguish between strain bursts, pillar bursts, and fault-slip bursts. Strain bursts are defined as sudden and violent failure of rocks near an excavation boundary with the bulking and energy for the seismic activity being co-located. Strain bursts are induced close to the excavation boundary and are further categorised as:

- Mining-induced strain bursts seismicity generated by the rock breakage at the locus of the bursting.
- Seismically-triggered strain bursts seismicity generated remote from the excavation boundary.
- Seismically-triggered, dynamically loaded strain bursts there is an additional permanent deformation applied to the excavation.

A simple criterion used to assess the likelihood of strain bursts is the spalling criterion, which is the ratio of the deviatoric stress to the uniaxial compressive strength. Values of between 0.6 and 0.8 are often used to identify the potential for strain bursts

Terminology

In the following discussion the term bump will be used to refer to the seismic activity, and burst will refer to a sudden fall of ground with no reference to the velocity or violence of the movement. In fact it would be preferred that the use of the term "burst" was abandoned as it conjures up dynamic behaviours that may be misleading. There is no doubt that high velocity ejection is associated with outbursts and also some strain bursts, but the key concept should be the energy at the time of impact with a person. The principle of momentum conservation requires that for the same dynamic event a larger mass must be ejected with a lower velocity although the energy release will be the same. By analogy to excavation trench collapses, it is considered that there is enough gravitational potential energy in a coal rib to cause death or injury to persons or damage to equipment as a result of a slump.

In the absence of engineering controls (ground support), sudden falls of ground can result from strain bursts, pillar bursts, and fault slip bursts as well as from gas outbursts and kinematic failures (Figure 2). This paper will concentrate on the concepts of strain bursts in coal mine roadways and also on kinematic failures, extending the work of Seedsman (2006) to highlight the possibility of a kinematically acceptable block that may move without any stored or added strain energy but simply as a result of gravitational potential energy. It is argued that fault-slip bursts are relatively rare in Australian coal mines because of the general lack of faults in longwall mines such that it is unlikely that large expanses of extraction can remobilise faults that cut across roadways. It is recognised that pillar bursts are possible in thin fenders of coal that may be formed as a longwall approaches a predriven roadway. There can be pillar bursts (or longwall face bursts) as a result of extreme weighting events (Mark, 2014).



Figure 2: Mechanisms for sudden falls of unsupported ground: yellow arrows show possible seismic input, blue arrows show possible seismic emissions

Strata failure management plans seek to prevent falls of ground through the use of ground support which in an Australian coal mine is typically in the form of bolts and mesh. It is possible that this support is inadequate in the face of a seismic event – if so this is referred to as seismic shakedown.

SEISMIC SOURCES

Seismicity in coal mines has been extensively studied (Kelly and Gale, 1999) however large magnitude seismic events have not been a characteristic of longwall mining in Australia. There are reports of a ML4.5 earthquake in the coal mining area around Wollongong (ANSIR, 2000) and there is hearsay that ML3.0 events have been recorded in the Cessnock district of NSW. The 2007 collapse at Crandall Canyon was associated with a 3.9 ML event (MSHA, 2008). Swanson *et al* (2008) refer to experience in western US coal mines where seismic events of 2 ML and 3 ML can occur without any noticeable impact to mining operations or even an awareness that such events have occurred. They report two case studies where floor heave and rib spall were associated with 1.9 ML and 2.9 ML events and in both cases the damage was localised to areas of steeply dipping faults with no evidence of fresh macroscopic slip movement on the fault planes.

Of greater interest in the context of managing the impacts of seismic activity is how such activity is felt underground. Compared to the state-of-the-art in deep metal mines which have sophisticated seismic monitoring and a design process based on peak ground velocities, there is little information on seismicity in Australian coal mines. This is partly due to the generally lower seismic activity and possibly partly due to the lack of reported damage or impact to the coal mine roadways. In lieu of such information, Table 1 is focussed on suggesting an intensity scale by combining qualitative descriptions presented by ACG (2008) with the Mercalli scale as downloaded from Wikipedia. By reference to Table 1, the author has experienced intensity IV and V events in some of the longwall mines in both NSW and Queensland and was on the surface for one VI event. For the USA mines, Hoelle (2016) recalls bumps being mostly intensity V and one event with intensity VI; he recalled lots of sloughing from the (unsupported) ribs.

Qualitative Description – underground metal mines (ACG 2008)	Mercalli – Earthquake effects on the surface	Mercalli Perceived shaking	Mercalli Peak Ground Acceleration (g)
Ground shaking felt close to the event. Felt as good thumps or rumbles. May be felt remotely from the source event (more than 100 metres away). Often detectable by a microseismic monitoring system.	IV – Felt indoors, dishes, windows, doors disturbed. Walls make cracking sound. Standing motor cars rocked noticeably	Light	0.039g
Often felt by many workers throughout the mine. Should be detectable by a seismic monitoring system. Significant ground shaking felt close to the event.	V – Felt by nearly everyone, Some dishes or windows broken, Unstable objects overturned.	Moderate	0.092g
Vibration felt and heard throughout the mine. Bump may be felt on surface (hundreds of metres away), but may audible on surface. Vibrations felt on surface similar to those generated by a development round.	VI - Felt by all, Some heavy furniture moved.	Strong	0.18g
Felt and heard very clearly on surface. Vibrations felt on surface similar to a large production blast. Events may be detected by regional seismological sensors located hundreds of kilometres away.	VII - Considerable damage to poorly built or badly design structures. Some chimneys broken. People loose balance	Very strong	0.34g
Vibration felt on surface is greater than large production blasts. National seismic stations can usually detect events of this size.	VIII - Fall of chimneys, factor stacks, monuments, walls. Heavy furniture overturned.	Severe	0.65g

Table 1: A suggested correlation between mine observations and the Mercalli intensity scale

The basic mechanisms proposed for rock bursts (Figure 3) may not apply to coal longwall mines because of the presence of jointing in the sedimentary sequence and the lack of large-scale faulting. Coal-mine microseismics studies have located events mainly in the roof with a few in the floor. Recent Australian coal industry research has sought to explain the seismicity by examining possible fracturing

events ahead of the longwall face in a two dimensional plane drawn along the centreline of a longwall extraction panel (Medhurst *et al*, 2014). This approach relies on seismic events generated by bedding-parallel shear and the onset of tensile fractures as progressive cantilevers fail (Figure 4). However, this approach does not consider the pre-mining presence of joints and cannot consider the possibility that the overburden rocks may span out of the plane of the cross-section and across the extraction panel.

It is suggested that a better model may be the failure of jointed rock beams that cannot span as the dimensions of the extraction panel increase. This mechanism can be considered in two dimensions if the analysis section is drawn parallel to the dominant joint set, which may or may not be parallel to the face line. Figure 5 shows the simplifed model of how a voussoir beam fails in compression and it is noteworthy that in laboratory-scale physical models such failure initiated by crushing at mid-span at the top of the beam (Sterling 1980, Passaris *et al* 1993). Also shown in Figure 5 is a typical relationship between the span of a voussoir beam and the required thickness to span. Any research into the actual seismic energy that could be developed from the compressive failure of a 20 m to 30 m thick voussoir beam has not been located.



Figure 3: Basic mechanisms for mine seismicity (Hasegawa *et al* 1989) and the location of seismicity above a longwall extraction panel (Kelly and Gale 1999)



Figure 4: Model for longwall seismicity invoking shear along bedding and new tensile fractures compared to a model of a pre-existing jointed rock mass



Figure 5: Typical thicknesses of a marginally stable voussoir beam as a function of span

In summary, it is assessed that coal mine bumps are of low seismic magnitude and most are associated with the longwall caving mechanism and not the reactivation of faults. The intensity of the shaking felt underground suggests that horizontal accelerations of up to 0.2 g may be typical. It is speculated that if the failing voussoir beam is located very close to the coal seam, the subsequent collapse of the released joint blocks may be the soft loading system required for a pillar burst or a seismically triggered dynamically loaded strain burst. Such a mechanism could also be applied to the longwall face bursts discussed by Mark (2014).

STRAIN BURSTS AND THE INNER SHELL OF COAL MINE ROADWAYS

Seedsman (2014) has applied the concepts initially proposed Martin *et al* (1999) to predict the depth of brittle failure around coal mine openings and incorporating the impact of transverse isotropy. To address the lack of a suitable failure criterion that incorporates both the brittle and spalling components for the inner shell (Kaiser *et al*, 2015), the depth of failure around coal mine roadways is considered to be the lesser of two rings identified in the Transversely Isotropic Brittle failure criterion (TIB) – a cohesion ring and a friction ring; reflecting the presence of jointing and cleating, the tensile strength is assumed to be zero. The cohesion ring is defined by a Mohr-Coulomb criterion with cohesion equal to UCS/6 and zero friction. For an empirically derived ratio of the Youngs Modulus to the independent shear modulus (E/G) of 15, the friction ring for stone is set at a σ^1/σ^3 ratio of 3.4 (equivalent to a friction angle = 33°). Buzzi *et al* (2015) suggests a σ^1/σ^3 ratio of 38 may apply to high strength thermal coal (equivalent friction angle = 72°) which has been used in the following analysis with the same modulus ratio of 15.

An example of predicting the thickness of the inner shell in a coal mine rib

For Australian underground coal mines the horizontal stresses are typically 1.5 to 2.0 times the vertical stress in stone roofs, and in the order of 0.5 times the vertical in the coal seams (Seedsman, 2004). For a coal seam with a UCS of 16 MPa, an immediate stone roof of 60 MPa, and a mine operating at 550 m depth, this implies a spalling criterion of about 0.3-0.5 in the roof and about 0.9 in the coal.

Figures 6 and 7 present the following:

- Extent of failure in the inner shell with the colour contours showing strength factors less than unity for the cohesion component, truncated by a stress ratio of 15:1 in stone and 38:1 in coal. A typical roof and rib support pattern is anchored well beyond the indicated failure zones.
- b. The sensitivity of the maximum thickness of the inner shell in the coal ribs to the assumed value of the stress ratio at either mid-height or 0.5 m from roof or floor. Note that for lower values of the stress ratio the thickness of the inner shell is still less than about 1.25 m even for a value of 10.
- c. The shape of the inner shell is limited by the cohesion ring, in this case a UCS of 16 MPa. This thin geometry is a result of the assumption regarding transverse isotropy and differs from that obtained if isotropic parameters are used. It is noted that unsupported ribs at low depth of cover show the formation of slabs (Figure 7a) and for supported ribs with slightly higher coal strength fracturing can be seen near the roof line (Figure 7b) corresponding to the low strength factors.
- d. The concept of cohesion and frictional rings allows the development of a simple design chart that relates the maximum thickness of the inner shell in the coal ribs to the ratio of the UCS to the applied vertical stress. The chart shows the difference in shell thickness between the isotropic and transverse isotropic assumption. The frictional ring implies a maximum thickness of the inner shell regardless of the vertical stress if the UCS/vertical stress ratio (Coal Strength Index CSI) is less than 6. This is in agreement with experience that even at depths in excess of 750 m coal ribs look and behave similarly to those at shallower depth. Note that

the maximum thickness value depends on the assumption of the frictional stress ratio (Figure 6b). Care needs to be taken not to extend the lines in Figure 6d to very low CSI values.

Photos of typical rib conditions (Figure 7) provide support to the TIB predictions. As depth increases, the first signs of damage can be seen near the roof and the floor (Figure 7a) which is where failure in the cohesion ring first develops. If there is no ground support slabs can form, as predicted by the frictional ring, and then either slide or topple from the sides (Figure 7b). At greater depths, where the CSI value is low, the ribs start to collapse close to the mining face and before support can be installed (Figure 7c).

The containment of strain bursts

Interpreting the TIB failure in terms of strain bursts, it is suggested that the reported "spitting" of coal at the development face is associated with the early development of the cohesion ring. As the failure expands into the frictional ring, the coal will bulk, possibly in the order of 1%–3%, or between 10 mm – 30 mm if the failure extends 1 m into the rib. Such bulking can be adequately restrained by a flexible mesh, and it is noted that bolts and particularly mesh are considered to be critical controls for rock bursts in metal mines. The practicalities of continuous miners do not allow meshing near the floor while at the development face. Spitting at floor level probably represents a lesser hazard.



Figure 6: The transverse isotropic brittle (TIB) model for defining the inner shell about a coal mine roadway



(a) Onset of rib damage at top of supported rib



(c) Typical rib at depths greater than 500 m



(b) Slabs formed in an unsupported rib at shallow depth

Figure 7: Typical rib conditions showing the progression of damage as the ratio of the coal UCSto veritcal stress decreases

In summary, for a deep mine the coal ribs will undergo failure at the development face and before or very soon after when bolts and mesh can be installed. The weight of this failed coal that can potentially collapse is in the order of 3 tonnes/metre of roadway advance. Typically in Australian coal mines, bolts and mesh are routinely already used to manage the ribs and the bolt density used to adequately pin the mesh is well in excess of the dead-weight loading. In shallower mines rib deterioration is in the form of thin slabs (less than 200 mm thick). The question for coal mines then becomes whether the mesh and bolts are adequate if there is a subsequent seismic event, say associated with longwall caving. Kaiser and Cai (2013) consider that a seismic trigger does not increase the depth of failure; it requires additional imposed stain to increase the bulking and extend the depth such as would be associated with a pillar burst. Extending this further, it is possible that the imposed strain may interact with a slab geometry formed by the TIB failure and result in a buckling mechanism – a possible explanation for ejection in pillar bursts.

KINEMATIC FAILURES

Marginally-stable but kinematically-acceptable blocks that collapse under gravity may be formed either by the orientation of roadways to the joint/bedding structure (Seedsman, 2006) or by the onset of deep TIB failure in the coal such that slabs are formed. This section examines if seismic loading can induce such collapse and also examines if anchoring behind such blocks can prevent collapse. The following kinematic analyses simplify the rib to be monolithic blocks while in reality the collapsing ground would be defined by a number of sub-parallel joint structures such that a very blocky rill pile may be formed after the collapse.

Depending on the spacing and relative orientation of the discontinuities these kinematic mechanisms may dominate. At depth the relative slow stiffness of the jointed ground, will prevent it being loaded by

the full overburden stress. This possibly introduces the additional hazard of the workforce failing to recognize better ribs as being a precursor to more adverse ground.

Planar slides

Consider a 0.5 m thick column of coal, either defined by joints parallel to the roadway, or by brittle failure developed within the inner shell. The base of the column is defined by a fracture surface dipping at 25° with an equivalent friction angle of 30° . The factor of safety of such a column is 1.24 without seismic loading, reducing to 1.0 with a seismic loading of 0.08g (Figure 8). By reference to Table 1, this corresponds to moderate shaking - category V. When supported with the pattern shown in Figure 6a very high factors of safety apply, in fact even 1 bolt per metre would be adequate.



Figure 8: Stability of a supported or unsupported planar slide as a funtion of sesimic coefficient

Toppling

The coal rib geometry being considered is not directly compatible with many of the rock toppling analyses codes. By way of an approximation, joints dipping at 88° into the rib and a 0.3 m thick slab have been assumed (Figure 9). This slab has a factor of safety of 15 without seismic loading reducing to unity at a seismic loading of 0.08g. Bolting serves to increase the thickness of the column and hence increase its aspect ratio. Once the aspect ratio is greater than height * tan (cross dip) toppling is not kinematically possible.



Figure 9: Toppling of a near vertical slab as a function of the seismic coefficient

Wedges

Depending on the orientation of the joints sets, and especially if one of the sets has a moderate dip, large wedge hazards can exist in coal mine ribs. Consider a case where a coal mine roadway is driven through a complex strike-slip fault zone and finds itself at an angle to the strike of a shear surface. Experience is that shear surfaces in coal have friction angles of 15° to 20° and typically dip at about 40° to 45° . One possible 6 m deep wedge geometry, 12 m long and with a mass of 57 tonnes (Figure 10). For a 15° friction angle, the factor of safety of the wedge is 1.13, and at 20° friction the factor of safety is 1.32. Seismic loading reduces these factors to unity at 0.05g and 0.12g respectively, or shaking intensity V or VI (Table 1).



Figure 10: Stability of a wedge as a funtion of the sesimic coefficient

Figure 11 overlays the support pattern in Figure 6a onto the wedge geometry in Figure 10. Recognising the likely presence of multiple bedding partings within the coal, only the middle and lower bolts would be effective in supporting the lower portions of the wedge and, depending on the location along the face of the wedge, these two bolts may not be long enough to adequately anchor in stable ground.



Figure 11: Typical rib bolting pattern installed into the analysed wedge revealing insufficient length

MANAGING THE SEISMIC HAZARD

By drawing analogies with kinematic rock slope stability, collapse of ribs could be generated by bumps corresponding to level IV and V intensity. Such collapses would be sudden, but would not have violent ejection. It is noted the gravitational potential energy that could be released by a fall of 1 m³ of coal from 2.5 m is 40 KJ which would impact at the floor level at 7 m/sec (25 km/hour). The general lack of awareness of bursts in Australian coal mines may be related to the standard practice of securing ribs with bolts and mesh. The association of coal bursts with small scale faults in US coal

mines (Swanson *et al*, 2008) may be explained by the presence of non-vertical surfaces defining wedges and the absence of rib support.

A key conclusion of this paper is that the general observations of coal mine bursts being associated with massive units, elevated depth, and faults may be explained by the driving seismic mechanism (the bumps) being associated with failure of massive voussoir beams, the increasing depth relates to the increasing thickness of failed coal at the excavation boundary, and the faults defining kinematically acceptable wedges and other blocks. The use of words such as 'pressure' and 'violent' are not necessary as the gravitational potential energy within the blocks is sufficient to cause the observed damage and the terms may be misleading when devising a management plan.

It would appear that the key hazard to be managed in a coal mine strata control plan is seismic shakedown of either the excavation damaged zone within the inner shell or kinematically acceptable wedges within fault zones. Importantly this paper suggests that the shakedown hazards can be managed with bolts and mesh, with the important proviso that the bolt length is sufficient to anchor behind wedges in joint zones.

Based on this paper, key questions to address when developing a management plan are:

- Is it a caving operation? If not, there is unlikely to be a seismic hazard. If yes, it is important to note that the caving may be associated with active coal extraction or the caving may be somewhat delayed. The distance to the caving is possibly not material as the bedded nature of the overburden can provide wave guides to allow distant transmission of seismic energy.
- Are there thick overburden units, the caving of which may be delayed such that large seismic events are induced?
- Can the proposed shaking intensity table (Table 1) be used?
- Is the combination of coal strength and depth of cover such that a thick excavation damage zone will be created? At this stage Figure 6d can be used to give some indication of the likely thickness of damaged ribs, recognising that more research is required into the characterisation of coal.
- What system will be used to ensure that rogue structures that may define wedges and other blocks are identified at the mining face?
- Are there adequate barriers to protect the face crews before the bolts and mesh can be installed?
- Are we sure that our bolts and mesh panels are adequate for both inner shell support and seismic shakedown? It would appear that the Australian standard of rib support using bolt and mesh is adequate and should be the minimum standard for all longwall and pillar extraction mines regardless of depth if there is a possibility of delayed caving of massive units.
- What is the maximum practical length of rib tendons?
- Do we need to consider rib side protection along the full length of the miner?
- How will we manage the rib hazard outbye of the continuous miner? Can we invoke low spatial and temporal exposure probability to reduce the assessed risk?

Finally it is stressed that more research is needed on the proposed perceived shaking scale and its associated accelerations, the quantification of seismic intensity related to the collapse of voussoir beams, and the TIB failure criterion for defining the inner shell and how it can transition to the outer shell. In addition some of the concepts of TIB failure may assist in understanding pillar bursts.

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STUDY OF SEISMIC ACTIVITIES ASSOCIATED WITH AUSTRALIAN UNDERGROUND COAL MINING

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ABSTRACT: This paper reviews the seismic events that occurred within the New South Wales (NSW) mining regions in the past 10 years, using the seismic data obtained from Geoscience Australia. The frequency and magnitude of the seismic events were assessed to investigate the correlation between underground mining activities and the associated seismicity. The study also reviewed the seismic events associated with coal bursts in Australia to understand its nature and its proneness in Australian conditions. Based on the study conducted, there is no clear correlation between the past recorded seismic events and the underground coal mining activities. It is also suggested that the coal burst in Australia appears in low energy magnitude, occurring in isolated manner. In comparison to international experience, coal mines in China and United States have encountered significantly higher frequency and magnitude seismic events associated with coal burst. Based on the findings, it is recommended that localised seismic monitoring methods should be used to monitor low magnitude events with higher accuracy in regards to depth and location of events. The analysis and results produced from this study contribute to the knowledge and understanding of mining induced seismicity in Australian underground coal mines.

INTRODUCTION

Seismicity is a widely known phenomenon that is inevitable and associated with mining activities. Seismic events are associated with all types of rock failures. In underground mining, large seismic events can occur from various sources including pillar punching, disturbance of geological structure from active longwall mining, failure of overburden strata and events of coal bursts. A coal burst is defined as a pressure bump that actually causes consequent dynamic rock/coal failure in the vicinity of a mine opening, resulting in high velocity ejection of this broken/failed material into the mine opening. The energy levels, and hence velocities involved in pressure/coal burst can cause significant damage to, or destruction of conventional installed ground support elements such as bolts and mesh.

Despite of decades of research and experience, the source and mechanics of seismicity associated with coal bursts are inadequately understood (Mark and Gauna, 2015). The lack of understanding of such phenomena has made it difficult to implement effective coal burst control and mitigation measures. Seismic events associated with coal burst is regarded as one of the most dangerous hazards in coal mining which have accounted for considerable number of fatalities and substantial disruption to production capabilities (Westman *et al*, 2012). While there have been only two publically reported events of coal burst in Australia, coal burst events have been a serious prevalent issue in international mines located in countries such as China, Europe and United States. The burst events encountered by these mines have revealed that powerful seismic events associated with coal burst can occur due to various combinations of geological and mining operational factors such as deep depth of cover, inadequate pillar designs and presence of thick competent strata. This study aims to improve the understanding of the mining induced seismicity in Australian underground coal mines. The results will also form an important component in providing insights to seismic events associated with coal bursts in Australia.

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PAST SEISMIC EVENTS

As part of this study, comprehensive research into Australian and international coal mines affected by mining induced seismicity were conducted. The review of these mines involved examining the frequency and magnitude of the mining induced seismic event and assessing various factors that has contributed to the seismic activity. It also included comprehensive review of past coal burst events associated with these mines to gain improved understanding of the nature and casualties of these coal burst incidents.

Experience in United States

In the USA, events of coal burst and high magnitude of seismic activity have been closely associated with longwall mining. Study conducted by Arabasz and Pechmann (2001) has revealed that Utah has experienced high frequency of seismic activity induced by underground mining. By operating a regional seismic system, 148 mining induced seismic events were recorded between 1978 and 2000. These events have been identified with a local Magnitude (M_L) of greater than 2.5 with 18 occurring with magnitude greater than 3.0 M_I. In addition, the severe impacts of coal burst have been assessed, as 87 fatalities and 163 injuries were associated with coal burst within the database of 172 coal burst events recorded between 1936 and 1993 in the United States (lannacchione and Zelanko, 1995). According to Mark (2016), states of Utah and Colorado have experienced a series of fatal coal burst events that have had detrimental impacts on both safety and production of mining operation. Mine A located in Book Cliffs region experienced several powerful coal burst events throughout the life of mine. In response, various mitigation strategies such as interpanel barriers and abandoning of working panels were implemented. However, the persistent coal burst risks were not eliminated and claimed the life of a shearer operator when mining at a depth of 840m (Mark, 2016). Consequently, due to inadequate management of coal burst hazard with prevalent risks associated with continual mining at greater depth, the operation was terminated. Similarly to Utah, mine B located in Colorado also experienced severe seismic events associated with coal burst. Since 2009, the mine experienced several powerful coal burst events with three events identified with local Magnitudes (ML) of over 3 (Mark et al., 2012). These events led to extensive pillar failures and severe ventilation damage, eventually leading to the closure of the mine.

Experiences in Australia

Until the recent event of coal burst that occurred in Austar Mine in New South Wales, coal burst events were rare phenomena that posed minimal risk to Australian mines. According to the NSW Department of Industry (2014), the 2014 incident involving two fatalities experienced magnitude and volume of the pressure burst that "rendered the installed rib support ineffective". Prior to the 2014 coal burst event, Austar has experienced numerous coal bump events in the past. Coal bumps are different to coal bursts where it is a dynamic release of energy from intact rock failure that produces audible signals and ground vibrations. In the past, workers in Austar have accepted these audible sounds as normality associated with underground coal mining activity (NSW Department of Industry, 2014). It was also reported that prior to 24 hours of the coal burst incident, a powerful pressure bump was experienced at a location that was in close proximity of the incident scene (NSW Department of Industry, 2014). The pressure bursts were not identified as a risk as the geological conditions at Austar have not been previously encountered in Australia. Consequently, this has led to inadequate understanding of the sources and mechanisms of the coal burst incident.

Experience in China

In China, seismic events associated with underground coal mining activities have been a long existing prevalent issue, dating the first mining induced seismic event in 1933 at Shengli Coal Mine in Fushun City, Lianing Province (Li *et al.*, 2007). From a wide collection of coal mines in China, Mentougou Coal Mine in Beijing and the Fushun Coalfield in Liaoning Province are renowned for the significant

number of seismic event frequency. A total of 111,913 seismic events with a local magnitude of greater than $1.0M_L$ were recorded in the Mentougou Coal Mine between 1980 and 2000. The mine also experienced one of the strongest mining-induced seismic events during its mine life with an event magnitude of $4.2M_L$ detected in 1994 (Li *et al.*, 2007). In the Fushun Coalfield, 92,630 seismic events greater than $0M_L$ were detected between 1968 and 2005 with greatest number of activities recorded in 2001 with 7222 events (Li *et al.*, 2007). The mine also experienced a gradual increase of coal burst event in terms of its frequency and magnitude. Since 1990, there have been an increasing number of powerful coal bursts with magnitudes over $3.0M_L$. Through to 2005, a total of 86 strong coal burst events with magnitudes over 3.0 were experienced with largest event registering at a magnitude of $3.7M_L$ recorded in 2002 (Li *et al.*, 2007).

In China, the increasing depth of coal mining and its operational scale are becoming a serious concern. While only 32 coal mines were associated with coal burst events in 1985, there have been an increase to 142 mines that have experienced the burst event in 2012 (Wen *et al*, 2016). The significant growth of coal burst events are likely to coincide with the increasing depth of mining with more than 50 coal mines operating at a depth of more than 1000m between 2006 to 2013 (Wen *et al*, 2016). During this period, more than nine coal mines have experienced 35 powerful seismic events associated with coal burst resulting in 300 fatalities (Wen *et al*, 2016).

METHODOLOGY

In this study, seismic dataset was acquired from Geoscience Australia, consisting of parameters such as magnitude, depth, date, time and coordinates of the event. A monitoring period of 10 years was considered appropriate and sufficient for the scope of this study. The criteria considered for the dataset utilised for this study is summarised in Table 1**Table 1**. To gain a greater knowledge about the validity of the seismic data, direct enquiries were made to Geoscience Australia in regards to the accuracy of the produced results.

		·
Lo	ocation	New South Wales (NSW)

Table 1: Parameters used for seismic dataset produced from Geoscience Australia

Location	New South Wales (NSW)		
Recording Period	From 01/06/2016 to 01/06/2016		
Depth (km)	0 - 1000		
Magnitude (M _L)	0 – 9		

To represent the relevance and correlation of the seismic data with underground coal mining, the seismic dataset was plotted with overlays consisting of mining leases and operating coal mines in NSW as of 2014, as illustrated in Figure 1.



Figure 1: Overlay of NSW of coal mining district

The seismic dataset in NSW was closely reviewed to isolate the events that have occurred on or in close proximity of the coal mining region. To represent a greater relevance of seismic data for the scope of the study, the depth of these events were subsequently reviewed. As underground coal mines in NSW operates within a depth of 1km, the "0km" events expressed by Geoscience Australia was considered relevant for this study. The updated sets of seismic data produced from these criteria were plotted on Google Earth software to examine the possible correlation between seismic activity and underground coal mining. From this process, details such as clusters of seismic events and event magnitude were analysed to gain an understanding of seismic activities in NSW coal mining districts. In addition, past coal burst events were assessed and enquired to respective mining companies to draw a correlation between recorded seismicity and reported coal burst incidents.

RESULTS

Assessment of seismic events in NSW coal mining district

Within the monitoring period of 10 years, 699 seismic events were recorded in NSW. The assessment of these events has revealed that 80 events were present on or in close proximity of NSW coal mining district. As seen in Figure 2, the majority of seismic events occurred within a depth of one kilometer, followed by considerable number of events occurring at depth of 10 kilometer and deeper. While the "0km" events can be attributed to human activities such as commercialised surface blasting and mining activity, events occurring at significantly deeper depth at around 10 kilometer are likely influenced by natural causes. According to the USGS earthquake magnitude scale, majority of these events were low energy events with vast number of them occurring at a magnitude between 2 to 2.6, as shown in Figure 3.



Figure 2: Varying depth of seismic events that has occurred in NSW coal mining district in the past ten years





Assessment of low depth seismic events

For the scope of this study, seismic events that have occurred within the depth of one kilometer were focused. Out of the 80 events that were presented on or in close proximity of NSW coal mining districts, 28 events have occurred within the depth of one kilometer. The magnitudes of these events are presented in Figure 4 and the associated locations of these events are shown in Figure 5.



Figure 4: Magnitude of seismic events occurring within the depth of one kilometer



Figure 5: Location of seismic events within a depth of one kilometer presented in NSW coal mining district.

Figure 4 reveals that vast majority of the "0km" depth events have occurred at a low magnitude manner with an average of $2.3M_L$. In addition, observation made from Figure 5 reveals a relatively well spread of events within given monitoring years with slightly more seismic events occurring between 2013 and 2014. To provide clarity in the particular regions with cluster of seismic events, a closer view of these districts is provided in Figure 6 and Figure 7. It is evident that there have been clusters of events occurring in Austar and Appin districts during the period of between 2011 to 2012 and 2013 to 2014, respectively. Relative to Austar, Appin has experienced slightly greater intensity of events with strongest event registering at a magnitude of $3.5M_L$. In addition to this, it is evident that the other mining district, which may indicate a common source of causality.


Figure 6: Close up view of Appin mining region



Figure 7: Close up view of seismic events in Austar mining region

Investigation of seismic events

With past seismic events associated with coal burst reported in areas such as Austar and Appin, these two districts were selected for further investigation. From the seismic data produced from Geoscience Australia, seismic activity on the 15th of April 2014 was searched, the day when two workers were killed due to a coal burst event. Although the magnitude of the event was described as a powerful event by NSW Department of Industry (2014), no seismic data was detected on that day or in proximity of the incident date. Similarly, coal burst event reported in Appin region also resulted in the absence of data recorded by Geoscience Australia. This was followed by conducting a direct enquiry to the mine operators in Appin and Austar about the seismic events that was recorded in close proximity of their underground coal mining operations. The enquiry consisting of event location, magnitude and time was conducted, however it was responded that no event of coal burst were present on neither the given details nor any reports of mining related incidents/events. Due to the lack of seismic event data, a clear correlation between underground mining and seismicity could not be found.

ANALYSIS AND DISCUSSION

Comparison with overseas experience

The observation of the seismic events in NSW mining districts revealed that Australian mines experiences significantly lower frequency of seismic activity compared to international coal mines. It has also suggested that the coal burst events in Australia appear in low energy magnitude that occurs in an isolated manner. The seismic data produced from Geoscience Australia has indicated that only 28 potentially relevant mining induced seismic events were recorded within the NSW coal mining districts in the past 10 years.

Furthermore, coal burst event has been a rare phenomenon in the Australian coal mining environment. In comparison to coal mines in United States and China, mining induced seismicity have been a persistent challenge with several presence of events with magnitude exceeding $3M_L$. Consequently, the impacts associated with these international coal burst events were considerably

greater than of Australian experiences. Previously, coal mines in United States and China has experienced powerful seismic activities associated with coal burst that has led to severe ground failures. In United States, coal mines in Utah and Colorado has experienced collection of severe and persistent coal burst hazard in the past which ultimately led to the closure of the mine.

From the assessment of mining induced seismic activity from both Australian and international coal mines, it was revealed that the frequency and magnitude of seismic events are highly dependent on the depth of mining operation. In Australia, mining activities are relatively shallow with Austar mine reportedly to be operating at a depth of 555m when the coal burst incident occurred (NSW Department of Industry, 2014). In comparison, coal mines in Utah and Colorado that has experienced devastating impacts of coal bursts were reported to be operating at a much greater depth, reaching up to 840m and 800m, respectively (Mark, 2016). Similarly, the collection of vast examples of severe coal burst in Chinese coal mines are highly attributed to significant mining depth with some Chinese mines operating at double the depth of some Australian coal mines. Ultimately, the collective experience of coal burst in Australia, United States and China has revealed that despite decades of research and experience, coal bursts events are difficult challenge to be adequately managed.

Limitation of seismic data

The lack of correlation between seismicity and underground coal mining activity can be highly attributed to the selected method of acquiring seismic data.

For depth of seismic events, Geoscience Australia has identified details of the depth parameter with a fixed value. For example, for an event that has occurred within the depth of 400 meter from the surface, Geoscience Australia produced this event as "0km" seismic event. The vague nature of this parameter can be a large problematic factor. Categorising the depth of events with a fixed value makes it highly difficult to identify the casualty behind the seismic event. If a more specified and accurate indication of event depth is given, for example as 300m or 400m depth events, it could be predicted with relatively high confidence that the seismic event has been influenced by mining related activity.

The use of seismic data produced from nationwide seismic monitoring stations such as Geoscience Australia can lead to a degree of inaccuracy when determining the location of seismic events. While the location of a seismic event can only be identified within a few tenths of a kilometre, it is possible that poorly located events can have a significant variance between the detected and actual event location. This can greatly affect the validity and accuracy of the analysed results. To resolve this issue, it is highly recommended that seismic records from regional seismic stations are used to locate the events with lower degree of uncertainty at a higher confidence level.

Another issue associated with using a nationwide seismic monitoring station is the inability to detect and monitor low energy magnitude events. Referring to Figure 3, there was relatively few low magnitude events recorded with lowest event being a magnitude of 1.4M_L. The lack of these low energy events may indicate that the monitoring stations used by Geoscience Australia have difficulties in detecting low magnitude events. This is also supported by the absence of detected seismic data of the coal burst event that occurred at Austar mining region on the 15th of April 2014. The lack of sensitivity of the monitoring system also indicates that microseismic events associated with mining activity cannot be detected. It is crucial that more sensitive seismic detection methods are employed to monitor smaller energy event. For example, in Utah a coal burst event registering at a magnitude of 1.1 was detected by using a regional seismic station (Mark, 2016)

CONCLUSION

This study analysed the seismic data produced by Geoscience Australia to assess the correlation between seismicity and underground coal mining in Australia. In particular, the study focused on the

relationship between coal burst events and seismic activity. The overview of seismic events in the past 10 years that have occurred in New South Wales coal mining district has revealed that there is no correlation between seismicity and underground coal mining activity.

From the assessment of past coal burst events and seismic dataset produced from Geoscience Australia, it was suggest that coal burst in Australia appear in low energy magnitude, occurring in an isolated manner. In addition, based on the direct enquiries made to Appin and Austar in regards to past seismic events recorded in close proximity of their mining district, no correlation was found with underground mining activity.

The study also compared coal burst and mining induced seismicity between coal mines in Australia, United States and China. Compared to Australia, underground mines in China and United State encountered significantly larger frequency and magnitudes of seismic events associated with coal burst. While similar geological characteristics were encountered between these mines, the significantly greater depths of mining in international mines are highly attributed to increased seismic activity associated with coal burst.

The approach undertaken for this study has certain limitations. The seismic dataset produced from national seismic scale involved uncertainties in regards to depth and location of the events. In addition, the detection of small energy or microseismic events was proven to be highly difficult by using the chosen seismic monitoring method. Due to these limiting factors, a clear correlation between seismicity and underground mining could not be established. Upon the completion of this study, it is highly advisable that localised seismic stations are used for future studies.

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A PRELIMINARY STUDY OF DYNAMIC FAILURES IN LABORATORY TESTS

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ABSTRACT: Sudden and dynamic failure of rock/coal mass during mining is a serious threat to safety in underground mines. This failure is often referred to as 'rockbursts' or 'coal bursts', mainly attributable to high level of stress. In order to investigate the coal burst phenomenon, a series of laboratory tests were conducted to examine the failure patterns associated with a burst event. Optical glass cube samples were drilled under varying stress conditions, to investigate the influence of stress environment on the dynamic failure. The outcomes of these laboratory tests will improve the understanding of the loading mechanisms leading to coal burst, especially the influence of high stress environment.

INTRODUCTION

Coal bursts are a major threat to mining safety in underground mines, especially for workings at great depth. Potvin and Wesseloo (2013) stated that the possibility of experiencing a seismic event resulting in fatalities has arguably become the most important financial (and safety) risk in underground hard rock mines operating in developed countries.

In traditional laboratory tests of dynamic failures of rock/coal samples, it is necessary to stop the test and unload the sample in order to observe the failure patterns. However, the changes that occur within the sample during the unloading process are unknown and cannot be controlled. Accordingly, there is a need for an improved methodology to directly observe dynamic failures under high static loading. The objective of this paper is to study dynamic failures by replacing coal with optical glass in drilling tests (to simulate the mining process) for direct observation of the associated failure patterns by taking the advantage of the high transparency of optical glass. This paper describes the laboratory experiments which simulate the dynamic failures in optical glass by drilling optical glass cubes of 50 mm side length under static loading. The process of dynamic loading is filmed and the tested samples are photographed for further analyses. It is not the intention of this study to extend the findings of this study into *in situ* behaviour of coal burst but rather make observations of some of the established loading and failure mechanisms.

CAUSES OF COAL BURST

Geologic factors

Over the years many studies have been conducted into the mechanism of coal bursts. Although the exact causes have never been understood (with any confidence); the following geological factors have been identified as contributors of coal burst by combining *in situ* observations and measurements with computer back-analysis:

- Depth of cover (Holland, 1958; Maleki, 1995; Makeli *et al*, 1999)
- Sandstone channels (Hoelle, 2008; Agapito and Goodrich, 1999; Maleki et al, 2011)
- Seam rolling and pitching (lannacchione and Zelanko, 1995; Maleki et al, 2011)
- Faults (Holland, 1958; Holub, 1997; Agapito and Goodrich, 1999; Alber, *et al*, 2008; Swanson *et al*, 2008)

With respect to the depth of cover, the self-weight of the overburden strata is a source of high static stress, which is considered to be the most critical factor in the occurrence of coal bursts. According to

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Agapito and Goodrich (1999), a rule of thumb used in Utah coal mines is that coal burst problems start at a depth of 450 m with strong immediate roof and floor.

Loading environment and loss of confinement

Essentially, coal is capable of bearing high vertical stress only with the existence of large confining stress. Under such circumstance, non-violent yielding will normally occur when the vertical load is increased to its peak strength, which is the case in a triaxial test. However, during mining or other off-seam seismic events, this confining stress is dissipated suddenly and the strain energy stored in the coal seam is released in a violent manner. Babcock and Bickel (1984) concluded that coal can be made to burst given necessary conditions of stress and constraint. In cases where the strength is largely produced by constraint, the sudden loss of this constraint can initiate the burst. The proposed testing programme somewhat simulates the loss of confinement in a sample by drilling into it.

TEST SAMPLES AND SETUP

Sample material selection

Two types of materials were tested in this project: Poly Methyl Methacrylate (PMMA) and K9 optical glass. PMMA is also known as acrylic based resin or Perspex. It is a thermoplastic of light weight and high transparency. It has a significantly high transmitting rate of visible light reaching 92% which is higher than normal glass. Currently, PMMA has been widely used as a substitute for transparent glass taking advantage of its high transparency and mechanical strength. K9 glass is a type of optical borosilicate crown glass manufactured in China. It is normally used in making the prism and optical lenses due to its low cost, high refractive index and high clarity. These two kinds of material are the only two materials that can be purchased from the market satisfying experimental requirements in terms of transparency, sample size, mechanical strength and cost. Semi-transparent materials are not considered in this project due to their influence on fracture observation. Figure 1 shows a comparison between the samples of PMMA (left) and K9 optical glass (right).

Under high levels of loading in a Uniaxial Compressive Strength (UCS) test, plastic deformation is observed for PMMA material in 100 mm size. K9 optical glass showed brittle characteristics that are similar to coal and consequently was chosen to be the material for this project. Figure 2 shows the PMMA sample (left) after a UCS test undertaken in the MTS rock testing machine.





Figure 1: PMMA (left) and K9 glass (right)



Sample size selection

Two different sample sizes of K9 glass, namely 50 mm and 100 mm (in the shape of cubes as illustrated in Figure 3), were evaluated before the experiments. In the UCS test of 100 mm sample, the peak strength of the sample exceeded the limit of the hydraulic loading cell in the MTS testing machine. Therefore, 50mm samples were used in all experiments, which quadruple the stress at the same magnitude of loading.

Figure 4 illustrates the 50 mm sample after a UCS test.



Figure 3: 50 mm sample (left) and 100 mm sample (right) Figure 4: 50 mm sample after UCS test

MTS rock testing machine

The UCS test is conducted using the MTS rock testing machine, as shown in Figure 5. A consistent loading rate of 0.1mm/s is used in all tests.



Figure 5: MTS rock mechanics test system

Drill and hydraulic press

The drill used in the final test is illustrated in Figure 6. The magnetic drill is a specialised power tool used in the drilling of structural steel. It has a strong electromagnetic base enabling it to adhere to a steel surface. The magnetic drill offers increased stability and also provides better accuracy.



Figure 6: Magnetic drill and hydraulic press

Camera

A Sony A6000 camera is used in this project to record the fracturing process within the sample during drilling. In slow motion mode, this camera is able to film 720p footage at a rate of 50 frames per second, which is capable of recording the fast propagation of cracks within samples. A higher speed camera was also evaluated for these experiments. However, due to length of each experiment, the data storage capability of available cameras was exceeded.

TEST PROCEDURE

UCS test of intact samples

An intact optical glass sample was first tested to obtain its UCS value prior to drilling tests, as illustrated in Figure 7. In order to ensure the correct loading of the samples, a spherical-seat was used in these experiments as further preparation of samples was impossible. As indicated above, a consistent loading rate of 0.1 mm/s is used in the tests. The results from these tests are summarised in the following sections.



Figure 7: UCS test of intact sample

UCS test of drilled samples

The second stage of testing was UCS testing of pre-drilled samples. The purpose of the second stage testing was (i) to observe the crack propagation around the borehole and (ii) to determine that the drilled samples fail before they reached the maximum capacity of the hydraulic pump that is used to provide the static loading of samples. The maximum capacity of the pump was 200 kN. The pre-drilled borehole is 12 mm in diameter and 40 mm long. Figure 8 illustrates a drilled 50mm sample in a UCS test.



Figure 8: UCS test of a drilled sample

Loaded drilling test

The tests were undertaken in two different testing frames. The first frame is for 50 mm samples and the second one is for 50 mm x100 mm rectangular samples. In the first frame (Figure 9), the magnetic drill is attached to top steel panel and the drill points to the sample centre. The samples are preloaded with different magnitudes of loads starting from 10 kN.



Figure 9: Final test platform for 50 mm sample

The second drill frame is shown in Figure 10. The reason for using a different frame is the location of the drill in relation to sample location. When longer samples were used in the first frame the drill bit did not target the middle of the samples; which resulted in imbalance loading of the samples during testing; therefore, the second frame was used for 50 mm x 100 mm rectengular samples.



Figure 10: Final test platform for 100 mm sample

TEST RESULTS

UCS test results of intact samples

Figure 11 shows the load – displacement curve of a50 mm sample during a UCS test. The graph indicates that the UCS of the sample is approximately 420 MPa. During the tests it was observed that once the sample reached its maximum strength (and the initial failure occurred) further loading of the sample was possible. However, due to safety reasons the tests were stopped once the samples were loaded up to the maximum strength. Therefore, no post-peak data is available from these tests. It is of note that the UCS of optical glass is not an appropriate parameter to predict cracking in this project as the glass starts to crack at a lower stress which is around 300 MPa.



Figure 11: UCS test result of 50 mm sample

UCS Test Results of Drilled Samples

The samples used in this test are somewhat different from the intact sample tests presented above. Figure 12 shows a drilled 50 mm sample under loading of 100 kN. The graph indicates that the stress is concentrated around the borehole and leads to the initiation of cracking when the load is increased. As a result, the severity of cracking reveals the level of stress at a particular point.



Figure 12: 50 mm sample under 100 kN loading

Figure 13 illustrates a comparison of 50 mm and 100 mm drilled samples. The drill depths of each sample are 40 mm and 50 mm respectively. The 100 mm sample starts to fracture at a much higher stress during the UCS test which exceeds the limit of hydraulic press and consequently, 50 mm is chosen to be the size of test sample for the final tests.



Figure 13: 50 mm and 100 mm sample

LOADED DRILLING TEST RESULT

A number of issues arose during the loading of intact samples and these issues cannot be neglected when the magnitude of loading exceeds 160 kN. The issues are summarised as following:

- Point loading on sample surface due to uneven surface of seating
- Irregular increased loading due to manually pumped hydraulic press
- Non-vertical loading due to tilted testing frame

All these issues result in the same problem that the sample fractures to some extent before the drilling stage. The pre-existing cracks influence the occurrence of dynamic failure negatively in such a way that progressive stress induced failure occurs instead of a dynamic failure. Among all of the tests, only four tests were successful with no cracks in the sample before drilling. The uniaxial loading of these four tests are 40 kN, 70 kN, 130 kN and 160 kN respectively. During the final tests, the camera recorded the process of drilling until the occurrence of burst. In order to keep the sample intact for analysis, drilling was stopped once the burst occurred and photos were taken. The result of each test is demonstrated in four different directions: front view, back view, side view and top view.

Test result at 10 kN

Figure 14 illustrates the test results under 10 kN loading. As clearly shown in the figure, there is no sign of failure, despite the drill reaching the end of the sample. Less than 10 kN loading, the stress on the testing sample is equal to approximately 4 MPa, which is much smaller than the maximum UCS of 420 MPa. This test result indicates the fact that drilling itself will not cause a dynamic failure or fracturing of the sample.









Figure 14: Test result under 10 kN in (a) front view (b) back view (c) side view (d) top view

Test result at 40 kN

(c)

Figure 15 illustrates the test result under 40 kN loading. Different from the 10 kN loading, it can be seen from Figure 15(b) that extra damage has occurred to the sample when the sample is drilled through along the direction of loading. However, this cannot be classified as burst because of the gradual failure process during drilling as reviewed in the video.



Figure 15: Test result under 40 kN in (a) front view (b) back view (c) side view (d) top view

Test result at 70 kN

Figure 16 illustrates the test result under 70kN of static loading. Under the 70kN loading, dynamic failure occurs before the drill reaches the end of a sample. As can be seen from Figure 16(b), the cracks caused by dynamic failure concentrated around the top of the borehole and radiate out along the direction of uniaxial loading. As is also evident in Figure 16(c) there was no crack around the borehole until dynamic failure occurred.



Figure 16: Test result under 70 kN in (a) front view (b) back view (c) side view (d) top view

Due to the pre-existing cracks, tests under 50 kN and 60 kN failed and the test results were not strictly valid. However, it is reasonable to conclude that dynamic failures started occurring once the static

loading is increased to approximately 70 kN, which is approximately 7% of the maximum strength of the K9 glass.

Test result at 130 kN

Figure 17 illustrates the test result under 130 kN of static loading. Dynamic failure occurs before drill bit reaches the end of sample and the position is shown in Figure 17(c). Under a higher magnitude of loading, unlike the 70 kN test, the sample starts to crack around the drill hole during drilling as shown in Figure 17(c). This failure somewhat reduces the intensity of the dynamic failure.





Test result at 160 kN

Figure 18 illustrates the test result under 160 kN of loading. In this experiment dynamic failure occurred as the drilling started and reached the maximum intensity when the drill bit was approximately half way through the sample, which is earlier than the other loading cases.





ANALYSIS

In the analysis of test results, the following five factors were analysed using the sample pictures and videos:

- Stress level
- Burst position
- Timing
- Fracturing pattern
- Drill cutting

Stress level

As mentioned above, dynamic failure appears to occur at a static pre-loading of approximately 70 kN. The occurrence of a dynamic failure is significantly affected by pre-existing fractures in a way that the pre-existing crack propagates from surface to drill hole during drilling and prevents the sample from failing violently. In this project, only four tests under 40 kN, 70 kN, 130 kN and 160 kN loadings were conducted successfully without pre-existing cracks. When samples with pre-existing cracks were tested under loading higher than 70 kN, the sample failed in a progressive way as cracks initiated from the surface and then merge with the drill hole. The strain energy caused by high stress is gradually released in this process, which prevents occurrence of dynamic failure.

Dynamic burst position

Along with the high stress environment, another significant factor of dynamic failure is the location of the drill bit with respect to the sample size. Figure 19 illustrates the drill depth at the time of dynamic failure occurrences from the videos recorded. No dynamic failure occurred when the drill bit reached the other end of the sample at 40kN. When load was increased to 70kN in Figure 19(b), the dynamic failure occurred at the position which is nearly at the end of the sample. As loading increases to 130kN and 160kN, dynamic failure position gets closer to the drilling starting point. This can be explained by the fact that the load increases in the intact section of the sample during the drilling process (i.e., pinching of the load) and the higher the initial load the earlier the dynamic failure occurs.



(a) 40 kN (b) 70 kN (c) 130 kN (d) 160 kN Figure 19: Burst position within test samples

Timing

Another feature of dynamic failure is its sudden occurrence and the high velocity in crack propagation. Figure 20 is a screenshot of a test sample under 70 kN loading, immediately before the occurrence of the dynamic failure.



Figure 20: Test sample under 70 kN loading before burst

Figure 21 is the screenshot of the same test sample at the moment of dynamic failure. It can be seen that the time shown on both figures is identical, which indicates that the dynamic failure occurred in less than 0.02 seconds (recording rate of 50 frames per second).



Figure 21: Test sample bursting under 70 kN loading

Fracturing pattern

The analysis of the fracturing pattern is mainly focused on the following aspects:

- Level of concentration
- Fracturing pattern in vertical direction
- Fracturing pattern in horizontal direction

Figure 22 is the back view of the test sample under 70 kN loading. It can be seen from this figure that the fracturing is concentrated around the centre of drill hole and radiates out. The level of concentration is highest at the end of the borehole and reduces when propagating.



Figure 22: Back view of test sample under 70 kN loading

Figure 23 shows the back view of test sample under 130 kN loading. From these two figures it is evident that fracturing occurs along the vertical direction and there is no fracturing along the horizontal direction. The main reason for this pattern is the major principal stress. The sample fractures along the direction of loading and release the strain energy in that direction. As a result, all samples have a fracturing pattern along the vertical direction.



Figure 23: Back view of test sample under 130 kN loading.

Drill Cuttings

The drill cuttings of the optical glass without any loading are shown in Figure 24. These cuttings are collected during the drilling process for UCS test preparation. It is evident that the cuttings are mostly powdery fine particles.



Figure 24: Drilling cuttings without loading

Figure 25 illustrates the drill cuttings collected after the 70 kN pre-loading test. From these two figures it is evident that the cuttings become blocky in the pre-loading tests. These blocky cuttings are mainly formed for two reasons:

- The glass powders agglomerate and form a small block due to the water for dust suppression during the test.
- The glass powders cover glass fragments and then agglomerate to form a small block



Figure 25: Drilling cuttings after pre-loading test

Inside the drill holes were also observed following the tests. In none-dynamic failure boreholes, the walls were very smooth and the hole diameter was highly consistent. In the cases where a dynamic failure occurred, the walls of the drill holes were fractured and failed resulting in uneven borehole walls. Unfortunately it was impossible to photograph these observations.

CONCLUSION

As published by many authors in the past, the results from these experiments indicated that high stress is a contributing factor for stress driven dynamic failures. In the laboratory tests, dynamic failure does not occur until the loading reaches 70kN. The phenomenon of dynamic failure of the glass under high levels of uniaxial stress and loss of confinement during drilling is consistent with the finding of Babcock and Bickel (1984), which suggests that coals can be made to burst given necessary conditions of stress and constraint.

Another finding is that the dynamic failures occur in a short period of time which was less than 0.02 seconds in this testing environment. This conclusion validates the sudden characteristic of coal burst in its definition.

With respect to the fracturing pattern, cracking concentrates around the centre of drill hole and radiates out along the vertical direction which is the direction of major principal stress. This test result not only embodies the influence of major principal stress on coal burst but also validates the conclusion that high stress is a significant contributing factor for coal bursts.

It is also found that pre-existing cracks plays an important role in delaying or even eliminating the dynamic failures. During drilling, pre-existing cracks propagate rapidly towards the drill hole and lead to the stress being transferred along the crack. Strain energy therefore cannot be accumulated and the stress cannot be concentrated in front of the drill hole to trigger a dynamic failure. This observation can validate the effectiveness of destress drilling in coal burst prevention.

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MANAGING OCCUPATIONAL HEALTH IN THE MINING INDUSTRY

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ABSTRACT: With the recent resurgence of "black lung" detection in the Australian Coal Mining Industry, the spotlight has fallen onto the management of the health aspects of health and safety. Too often health is seen as being hard to manage because it may relate to chronic and/or extended exposure to harm. However proper application of risk management techniques including focussing on critical controls and the continued effectiveness of these controls works as well on occupational health issues as it does on safety issues. This paper will demonstrate that risk management can be successfully applied to health issues including respirable dust, fatigue and psychological impairment. The resurgence of 'black lung" will be used as a case study to underline the need to identify critical controls and institute processes to measure and maintain their effectiveness and not allow them to be eroded.

INTRODUCTION

This paper addresses the management of occupational health in the coal mining industry using exposure to respirable coal dust in underground mining operations as an example. The key to implementation of modern OHS legislation is the requirement to reduce health and safety risks to workers to as low as reasonably practicable and to an acceptable level. The legislation does not specify what an acceptable level of risk is. In some cases it does provide guidance through stipulating such things as the maximum acceptable exposure to a hazardous substance, such as respirable coal dust, to which a worker can be exposed. There is an emphasis on "Duty of Care" where the onus is on the mine operator to establish risk levels and provide a work environment in which employees are not exposed to unacceptable levels of risk. Information, instruction, training and supervision are provided. The duty of care is shared between employer and employee however the primary responsibility rests with the employer, who has the largest control over working conditions.

Safety management is more mature than health management (Figure 1), often due to the perception of risk (or lack thereof) surrounding health issues. Safety risks are generally more visible, and therefore more salient, whereas health risks are often not immediately visible. Safety consequences usually have an immediate impact whereas health consequences often have a latency period of many years. Adding to the complexity of the situation are confounding risk factors such as the multi-factorial nature of most occupational diseases, individual inherent health differences (e.g. predisposing medical conditions) and transiency within the workforce making it difficult to identify exposure and understand causal relationships.

In terms of respirable dust exposure, there have been significant numbers of exceedances of the occupational exposure standard since at least 1991. Bofinger, Cliff and Tiernan (1995) reported on personal and static respirable dust monitoring over the preceding three years at four longwall mines in Queensland. Static measurements indicated a trend for increasing dust concentrations as the distance from the maingate increased. Cliff and Kiz (2002) analysed personal respirable coal dust measurements recorded by each mine and the Department of Natural Resource and Mines up to mid-2001 for the 11 longwall mines in Queensland. Measurements exceeded the statutory eight-hour equivalent exposure standard in 15.6% of cases (see Figure 2). Most recently, in a presentation at the 2016 Queensland Mining Health and Safety Conference, Djukic and Gill (2016) indicated respirable dust exposure levels regularly exceeded acceptable levels across a number of mine sites measured

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between 2000 and 2016 (see Figure 3). The black tracer line represents the total production across all sites with operating longwalls.



Figure 1: MIRM Safety Maturity Chart (Foster and Hoult, 2013)



Figure 2: Operator category eight-hour equivalent mean respirable dust exposure (Cliff and Kizil 2002)

The health effects of long term exposure to coal dust can be significant. The inhalable dust fraction has been defined by ISO 7708 (AS3640-2004) and is the dust fraction of the airborne particles which are taken through the nose or mouth during breathing into the body. Inhalable dust is made up of all the dust sizes that can deposit throughout the respiratory tract, and includes dust which will deposit in the upper and lower airways of the respiratory tract and through mucocilary clearance mechanisms in the gastrointestinal tract. The larger particles deposit in the upper airways (nose and throat).



Figure 3: Estimate mean (MVUE) for longwall SEG 2000-2015 (Djukic and Gill, 2016)

The smaller particles can penetrate the upper airways and deposit in the lungs (thoracic fraction) and respirable finer particles can penetrate to the alveolar region or gas exchange region (respirable fraction). The Queensland Coal Mining Health and Safety Regulations 2001 limit respirable dust exposure to levels not exceeding 3.00 milligrams per cubic metre (mg/m³) of air during any shift. The potential of coal dust to cause pneumoconiosis has long been recognised, and is essentially linked to exposure to respirable dust.

WHY IS OCCUPATIONAL HEALTH DIFFICULT TO MANAGE?

It is generally well-recognised that occupational health is more difficult to manage than safety and is sometimes described as the "poor cousin of safety" in terms of the time and resources spent on health management and the cost associated with illness and disease resulting from occupational exposures (Hopkinson and Lunt 2014).

The most recently reported total costs of injury and illness in the mining industry in Australia are \$1280 million and \$1160 million respectively (Safe Work Australia 2015), indicating that injury is a slightly more significant cost, however it is also recognised that cases of work-related disease are under-reported in both workers compensation data and through ABS surveys of the workforce. (NOHSC 2000; Safe Work Australia 2015). Because some diseases have long latency periods (e.g. cancers and pneumoconioses) and others are difficult to link to occupational exposures (e.g. cardiovascular and respiratory diseases), workers' compensation data significantly under-represent the actual incidence of occupational diseases (Safe Work Australia 2014). There is no comprehensive system of surveillance for occupational disease or illnesses in the mining industry in Australia.

Further complexities for the management of occupational disease are the interaction between occupational exposures and lifestyle factors and the complication of individual issues that might make a person more vulnerable to dosage and exposure (e.g. effect of smoking or asthma on the results of exposure to dust). Unlike injury where there is usually a clear relationship between an incident and the workplace, most occupational diseases are multi-factorial in nature, with workplace exposures constituting one important part of the risk matrix.

Symptoms of occupational disease often do not manifest until after an employee has left the workplace or retired from work. Tracking a person once they have left the work place is difficult and costly – however, it can be done.

A good example is Health Watch from the Australian Institute of Petroleum (Monash Centre for Occupational and Environmental Health 2013). Follow-up of individuals is further complicated by changes in occupation over a work life and the lack of records of work history including that no occupation is recorded on the Australian National Death Index.

In safety there is a strong recognition of the advantages of leading indicators such as high potential incidents, in addition to lagging indicators, to demonstrate management of an issue. In occupational health, there remains a nearly total reliance on lagging indicators, such as the incidence of disease, to determine the effectiveness of the management of occupational health issues. The faulty logic of this is demonstrated by the current Queensland situation. The lag indicators used include x-rays which have been shown to be faulty in terms of the implementation, quality and diagnosis (Monash Centre for Occupational and Environmental Health 2016).

The health related data systems that could provide information on leading indicators are ineffective. They do not capture data on prevailing work environments which could be used as a lead indicator and would assist in establishing relationships with health outcomes. The time lag from exposure to manifestation of dust related disease, the limited avenues to address the disease once it has been diagnosed, the difficulties with diagnosis and trouble tracking individuals show that we need to be proactive in the management of occupational health issues.

USING THE BOW-TIE APPROACH TO MANAGE OCCUPATIONAL HEALTH ISSUES

The bow-tie approach is a method commonly used to assist in the selection of controls for managing risks. While the focus needs to be on preventive controls, for health related issues it is often placed on mitigating controls. Monitoring is often wrongly considered as a control measure rather than a tool to assess the adequacy of the control measures in place.

The bow-tie approach provides a visual representation of the barriers used to prevent an unwanted event and mitigate its consequences. The knot in the bow-tie is the unwanted event - the point at which control is lost (see Figure 4). To the left of the knot are the causes and preventive and controls (i.e. a fault tree) and to the right are the mitigating controls and consequences (i.e. an event tree). The inclusion of both types of controls, plus the visual nature of the outputs allows gaps in the application of controls to be more easily identified. A problematic situation whereby there is a reliance on mitigating controls to reduce the severity of harm is quickly detected. A more proactive approach would be characterised by robust levels of preventive and controls (on the left side of the bow-tie) that minimise the exposure of workers to a hazardous event. The inclusion of mitigating controls is important to reduce the severity of harm, but the ideal scenario is preventing the event from occurring in the first place. Seatbelts are a mitigating control that reduces harm for example, but they do not prevent the vehicle losing control; controls that address driver behaviour, road surfaces and fit-for-purpose vehicles, help to achieve this. Successive layers of barriers are required to safeguard workers from adverse events – as described by Reason (2000) in his 'Swiss Cheese' metaphor.

The bow-tie method is widely used by mining companies in Australia to assist them in the implementation of safer operations – generally within a risk management framework. The controls shown represent those currently recommended in literature, provided by government mining and non-mining agencies (including mining regulations, codes of practice, guidelines and safety bulletins) and those known to be used by the industry. Information was also accessed from RISKGATE, an Australian Coal Association Research Program (ACARP) funded website, which has bow-ties for 18 mining-related hazards (see www.RISKGATE.org, Kirsch *et al.* 2013). RISKGATE was developed

between 2010 and 2015 from information provided by industry experts. The 'dust in atmosphere' bowtie is within the RISKGATE Occupational Hygiene Topic.



Figure 4: A schematic of the bow-tie approach (Kirsch et al, 2013)

A bow-tie for managing risks associated with hazardous levels of dust in the underground coal environment was developed – and is shown in Figures 5 and 6. The bow-tie includes controls that are indicative of those being used by the industry. It is a framework to consider the current focus of controls – to better determine whether there has been an over-reliance on mitigating controls, such as medical surveillance rather than preventive controls. Another aim of this discussion was to evaluate the application of bow-tie analysis to health-related mining hazards. For a more definitive list of controls used to manage respirable dust see, for example, Aziz, Cram and Hewitt (2009).







Figure 6: Mitigating controls and consequences (right-side) of the hazardous levels of dust in underground coal atmosphere bow-tie

The initiating or unwanted event (i.e. knot of the bow-tie) is hazardous levels of dust in the underground coal environment. It refers to atmospheric concentrations of dust that have the potential to cause harm. The potential for harm results from a combination of the concentration and the duration of exposure. Sources of dust include coal and silica and other respirable minerals. In this paper controls that seek to eliminate or minimise the amount of dust generated have been designed as preventive controls, while those that reduce excessive exposure to harmful atmosphere as mitigating controls. Mitigating controls are methods that do not prevent the amount of atmospheric dust generated instead they focus on minimising hazardous dust exposure. One such control is respiratory protective equipment. A caution given by Aziz, Cram and Hewitt (2009) regarding personal protection, which could more broadly be applied to other mitigating controls is – "[they] should only be used as a last line of defence and must not take the place of prevention or dust suppression techniques" (p. 568). In other words, coal mine workers' exposure to respirable dust must be kept to an acceptable level and below the regulated limit.

Mining plans and procedures, such as ventilation plans and traffic management plans are not considered to be controls – but rather activities that support controls. It is considered to be better to identify controls according to the International Council on Mining and Mineral's new definition of a control, which is an act, object (engineered) or system (combination of act and object) intended to *directly* prevent or mitigate an unwanted event (ICMM, 2015; see also Hassall, *et al* 2015). Accordingly, dust monitoring is not considered to be a control, because in itself it does not prevent the generation of dust or mitigate the exposure to dust. Rather, timely and efficient monitoring is a means of verifying the performance of these controls.

Six threats were identified involving underground mining events/tasks where workers are present and dust suppression controls are required. These are:

- 1. Coal drying due to gas drainage
- 2. Dust generated during development face operations
- 3. Dust generated during *longwall operations*
- 4. Dust generated during second working operations
- 5. Dust generated on conveyors
- 6. Dust generated as a result of movement along roadways

The first threat relates to the drainage of gas (and moisture) from the coal seam that occurs prior to mining to remove methane and CO_2 . Rehydration moistens the dried coal, reducing the potential for dust emissions.

The other five threats refer to operating environments and activities – including cutting operations at the face (i.e. development, and for second workings and longwall mining methods), conveyors for coal transport and roadways for vehicle transport; that disperse high concentrations of respirable dust into the underground atmosphere. Most Queensland underground coal mines use longwall mining methods. Longwall mining is thought to give rise to four times as much dust as continuous mining, particularly when production rates (machine speeds) are high (Monash Centre for Occupational and Environmental Health 2016). In addition, bi-directional cutting can result in increased coal mine dust exposure for miners.

The preventive and controls used in the bow-tie can be generally grouped into five categories: proper use of water sprays, ventilation (including use of extraction fans), fit for purpose cutters, cutting practice (e.g. speed, direction), and dust suppressant in the water system. The exception is for those controls related to movement along roadways, which involve quality, type and watering of roadways.

Two consequences are identified: (1) excessive levels of coal dust in the lungs and (2) development of respiratory disease. These are considered separately as they represent different stages in the trajectory of respirable illness – the first consequence may not necessarily lead to the second, but where it does, prognosis often occurs many years after exposure. Methods to reduce the amount of time that workers are exposed to harmful levels of dust include remote control mining; rostering, task rotation and work practices (including the positioning of workers near/on equipment); the segregation of returns and respiratory protective equipment. Although medical surveillance (e.g. X-rays, spirometry) is shown as a control for both consequences, when used as a control for the first, it is primarily used for the detection of early stages of respiratory abnormalities consistent with coal mine dust lung disease, that in-turn is used to prompt follow-up, referral and intervention. In the second consequence medical surveillance transitions from detecting pre-clinical changes in respiratory health to detecting confirmed cases of Coal Worker's Pneumoconiosis (CWP), involving tertiary intervention to track and manage disease progression.

This bow-tie does not address the impact of important individual factors (e.g. smoking, pre-existing lung capacity) that may confound the trajectory of illness and or cause one worker compared to another to be more vulnerable to adverse outcomes. Workers with multiple risk factors may be more vulnerable. This is difficult to capture in a bow-tie format. However, highlighting the role of individual factors can hinder the implementation of effective controls. For example, focussing on the individual, can lead to screening and recruiting of employees rather than a focus on preventing safety risks in the first place (see a more thorough discussion of a similar scenario in the Education and Health Standing Committee, Parliament of Western Australia (2015) report into the impact of FIFO work practices on mental health). Also, a recent finding of the Monash Review of Respiratory Component of the Coal Mine Workers' Health (2016) for the Queensland Department of Natural Resources and Mines found that there was a tendency for companies to attribute abnormal respiratory results on smoking rather than exposure to harmful levels of dust.

The bow-tie shown suggests that controls are available to address both prevention and mitigation of harmful levels of respirable dust in the underground coal mine environment. Recent confirmed cases of CWP in Queensland indicate that preventive controls have not been performing efficiently. An overconfidence in the current dust management system can perhaps be attributed to a reliance on the medical surveillance records of underground coal mining workers, which have until recently suggested the elimination of CWP. We now know that these health screening procedures have failed to effectively monitor the detection of respiratory abnormalities and CWP in workers (Monash report, 2016). A reliance on these measures to confirm the effectiveness of controls is also questioned, considering the potential long latency of disease. As highlighted in the previous section, the long-term

nature of health outcomes is problematic in terms of tracking workers over time, the follow-up of workers once they have left the industry, and the keeping of effective health data systems. Measuring control effectiveness has more recently become a focus in the management of risk in mining (Hassall, Joy, Doran and Punch, 2015; ICMM, 2015).

DISCUSSION

As the bow tie above illustrates, if workers are exposed to excessive respirable dust levels then either there are insufficient controls in place or they are ineffective. Given the plethora of research and advice on the subject there can be no reason for having insufficient controls. Controls fail to be effective because they are inherently inadequate or their effectiveness is eroded. Examples of control erosion include turning off or reducing the frequency of dust suppression water sprays because they impede the ability of the shearer driver to see where he is cutting. Lack of water pressure, poor maintenance of sprays and failing to change cutter picks often enough are all examples of factors that will erode the effectiveness of controls. A key factor in establishing control effectiveness is monitoring the implementation to not only ensure the controls are installed and operating as designed, but they actually achieve the desired level of control. Indeed monitoring must go beyond installation but extend to operation and maintenance. In these current days of cost control and production pressures it is easy to see where the continued operation of some dust suppression controls might be seen as negatively impacting on production. For example: production downtime whilst sprays are maintained. The current standard practice of monitoring exposure over a whole shift is not designed to detect sources of dust nor locations where a worker has been exposed to excessive dust levels. Real time dust monitoring is required if control effectiveness is to be monitored. Unfortunately there are currently very few real time monitoring devices approved for unrestricted use in Australian underground coal mines, though they have approval in the USA.

Because disease associated with excessive dust levels, like many occupational health issues, has a long term, cumulative impact, the immediate effect of ineffective controls is not obvious and can lead to a false sense of security, and a questioning of whether or not the control was really required in the first place. The apparent CWP free period of over twenty years may have led to a complacency and a questioning of the need to for dust controls other than Personal Protection equipment (PPE) and resulted in inadequate the exposure monitoring of the workforce. This complacency would be compounded by pressure from other areas of health and safety that have received attention in modern times, such as fatigue management, mental well-being, drug abuse and alcohol consumption – putting pressure on limited OHS budgets.

The bow-tie model demonstrates the need to manage respirable dust in a holistic manner. The NSW Work Health and Safety (Mines and Petroleum Sites) Regulation 2014 defines dust as an element of the principal hazard airborne contaminants and thus mandates the creation of a principal hazard management plan to control exposure to it. This is not a requirement in Queensland, though following the recent detection of CWP in Queensland coal miners, most Queensland underground coal mines have formed dust management committees and developed dust management plans.

Bow-ties can be applied to other occupational health issues, indeed using bow-ties to visualise the control process for occupational health issues illustrates just how complex managing these issues can be. RISKGATE (Kirsch *et al*, 2013) developed by coal industry working groups, contains within it bow ties for a wide range of health issues. The occupational hygiene topic contains bow- ties for respirable dust, diesel exhaust, hazardous substances, noise, heat and cold, vibration, asbestos and synthetic mineral fibres, waterborne contaminants and various kinds of radiation. The fitness for duty topic contains bow ties for alcohol consumption, misuse of drugs, physical fitness, fatigue, and mental ill-health. These bow-ties contain threats, preventive controls, consequences and mitigating controls. In some cases such as the mental ill-health topic, it is difficult to define controls in the traditional sense. It is also difficult to measure the effectiveness of these controls other than through the frequency of

the unwanted event or its consequences. Many of the controls will involve interaction with factors off the mine site and beyond the control of the mining company. The company can however influence the worker and the community behaviour through education and awareness and the provision of professional support and a positive work environment.

CONCLUSION

This paper has demonstrated, through the example of respirable dust exposure, that occupational health issues in Queensland underground coal mines have generally not been managed in the same way as other safety based hazards. The over reliance on mitigating controls, the lack of monitoring of the effectiveness of the controls and the mistaken reliance on medical examinations to indicate any adverse outcome has meant that CWP is still an issue in our coal mines.

By adopting a systematic risk management approach, incorporating techniques such as the bow tie methodology outlined above and paying proper attention to monitoring the effectiveness of controls, occupational health risks can be managed. Management needs to be approached in a holistic manner not piecemeal and not relying on PPE to protect the worker.

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IMPORTANCE OF ERGONOMIC APPLICATION FOR THE IMPROVEMENT OF COAL PRODUCTIVITY IN MINES

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ABSTRACT: The present study deals with the application of ergonomic intervention into a mining system with the view to increase its productivity. The effectiveness of the Ergonomic Work Rest Scheduling (EWRS) application, a vital tool of ergonomics, to reduce metabolic energy cost and increasing productivity with Effective Utilization of Man shift (EUMS) is the main focus of the study. Thirty five healthy underground trammers (N=35) were chosen based on some pre-determined criteria for the assessment. The personnel chosen from the mine were taken on the basis of experience of more than one year with no record of past illness. The study was performed in two different zones for one month of period each, one before application of EWRS and another after application of EWRS. Cardiac frequency of indirect physical parameters like Working Heart Rate (WHR) is recorded continuously at five seconds intermissions with a portable heart rate monitor. In case of Average Working Heart Rate (AWHR) value, it was seen that two different spells of after EWRS application value found lower than before EWRS application AWHR value. Different Cardiac Cost (CC) values like Net cardiac Cost (NCC) and RCC (Relative Cardiac Cost) also followed the trend of AWHR i.e. a significantly lower range after EWRS application. It was observed that application of appropriate EWRS has positively withdrawn the effect of the extra burden on mine workers. Perceived exertion scale had gone down to 13.91 as compared to before EWRS value of 15.11. It was also seen that application of EWRS effectively reduced the Net Metabolic Cost (NMC). The NMC of two different spells after application of EWRS seemed to be lower than that of Previous. After application of EWRS, NMC had gone down by 34.5% in the case of the spell 1 and 38.1% in the case of spell 2. Physiological job stress had been minimized by implementing adequate EWRS in between different working spells. Hence, it could be suggested that an implementation of EWRS and in depth ergonomic intervention would be very effective to minimize work-stress related problems and thereby increasing work efficiency and productivity of miners.

INTRODUCTION

Indian mining industries are one of the most powerful sectors which contribute towards the national energy demands since long. Indian mining industries have coal seams at such depth that it is not suitable for opencast mining and requires underground mining. In India the mining scenario differs from US and China (Singh R D, 1997). In a country like India, production from the manpower intensive underground mines is significantly low in comparison to open cast. The production from underground comes mainly from bord and pillar system (more than 90%). In deep mining, the room and pillar or bord and pillar method progresses along the seam. In the final stage of mining pillar extraction plays a major role. Apart from safety, the mining operations are also very hazardous and back breaking in nature (Dey *et al* 2015a; ILO, 2010). A cross-section on available literatures on Indian miners has configured some serious job-stress related problems (Dey and Sharma, 2013; Dey *et al*, 2014, 2015b). Various standing postures as well as higher job demand in mining job pattern makes the work process harmful (Sharma *et al*, 2016, Saha *et al*, 2010) which is accounting for lots of ineffective times and affecting production loss.

Not only the aforementioned problem associated with mining, injuries are also one of the potent causes of disruption in production (Pingle, 2016). Because of the increasing number of injuries

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caused by repetitive motion, excessive mechanical force, awkward postures, and application of Heavy Earth Moving Machineries (HEMM), ergonomics has become a most useful tool in the field of workplace safety. It is a process to look out an overall perspective for working layout in mining for suitable design of tasks, tools, equipment and most importantly fitting jobs to the miners. Body postures determine which posture of the body is going to be stressful and how much back strain will result for assuming awkward posture for a continuous time period. Spinal Discs mostly in the L5-S1 region is being affected when lifting, lowering or handling loads with the back bent or twisted for a continuous working shift. In the Indian mining industries there are many working groups like face drillers, roof bolters and dressers who are often used to work at or over shoulder height which could be stressful in particular. Implementation of ergonomically suitable postures with matching time exposure can reduce both the chances of injury and stress.

In fact; leading companies of the world are implementing agronomical measures into all of their operations to cope with the growing injuries along with the loss of production and productivity. Many of the national and international research studies have shown that application of ergonomics is really effective as far both the safety and productivity are concerned. The Washington State Department of Labor and Industries reviewed 250 ergonomic case studies to reveal the impact of ergonomics on business goals like cost savings, productivity and product quality (Middlesworth, 2016). From the previous research outcomes done on Applied Materials (supplier to the silicon chip industry) it was observed that properly designed and tested casters for manually moving 31.75 kN (7,000 lb) clean room manufacturing equipment increased productivity 400%, in terms of man hours, with reduced errors. Proactive Ergonomics culture in the industry emphasizes the deterrence of Work Related Musculoskeletal Disorders (WRMSDs) through recognizing, anticipating and reducing risk factors involved in a regular work culture reducing the chances of sickness absentees and herewith low productivity. Human factors and ergonomics are being implemented in the modern world as technological development focuses on the need of such requirements in daily work culture (Sanders and McCormick, 1993). Ergonomics application specially deals with five major improvement skills in industries. Many of the industries follow the beneficial goal point of these 5 skills and In this current research study application of EWRS and its effects on reducing energy cost and improvement in productivity is the main viewpoint.

Ergonomics has already been defined and its primary focus is on the design of continuous work activity that take place every day in mines. Fitting the job to the miners i.e. matching the requirements of a job with the capabilities of the worker with proper EWRS are the approaches to be adopted in order to lessen the risks of injuries resulting from non-ergonomic work culture in the present context of Indian mining. The present study deals with the application of ergonomic intervention to increase productivity in the industry. The initiative is taken by considering the EWRS introduction in between each and every working spell. The effectiveness of ergonomic EWRS application in mines to reduce metabolic energy cost and Cardiac Cost (CC) of miners are the main ergonomic viewpoints of the study. The goal of application of EWRS is to provide maximum productivity with minimal cost; in this context cost is expressed as the working Energy Expenditure (EE) of a worker. In a workplace setting there are seldom a large number of tasks that exceed the capabilities of most of the work force. There may be jobs that will include a specific task that requires extended reaches or overhead work that cannot be sustained for long periods, by using EWRS principles to design these tasks; more people may be able to perform the job without facing high EE demand and risk of injury.

METHODOLOGY AND INSTRUMENTATION

Description of mines

Study has been conducted in a mine of Coal India Limited in Jharkhand, India. The mine is of degree II gassiness which produces 1 m³ of inflammable gas per ton of coal produced having an inclination of 1 in 10 at a depth of 150 m. This mine has no shaft to descend and workers have to reach the working site by walking through the incline.

Selection of subjects

Thirty five healthy individuals (N=35) are chosen for the assessment. The proposed study was so schemed that a maximum gain can be achieved. The subject group is selected with the persons who were willing to participate in the study and after getting official permission from the Colliery Manager. The chosen personnel of underground trammer from the mine are taken on the basis of experience of more than 1 year. All the chosen workforce had no record of past illness.

Brief overview on underground tramming operation

Blasted and dressed coal is lifted into tubs (each one ton capacity) and dispatched to the surface via a haulage network. The empty tubs are supplied to the face by the trammers with the help of tugger haulage inside the working panel and loaded coal tubs are taken out from the underground panel and hauled up by the direct (main) rope haulage right up to the surface.

Principal Working Methodology

The working methodology is based on the improvement of productivity with application of the EWRS technique. Cardiac frequency of indirect physical parameters like Working Heart Rate (WHR) is recorded continuously in five second intermissions in two spells with a portable heart rate monitor (Sports Tester Polar Electro CS 400, Finland) by placing the machine on the trans-thoracic region of the subject before they go down to the working site. NCC and RCC are the most important derived parameters of WHRs which mirrors the cardiac strain intensity. The maximum heart rate of the subject is calculated by following the formula of American Heart Association (AHA, 1972). Determination of NMC (Ayoub, 1989) and EE (Datta and Ramanathan, 1969) are performed by the formula proposed by different scientists. In fact the study was performed in two different zones for one month each, one before application of EWRS and another one after application of EWRS. To fit with the perceived exertion scale, Borg physiological exertion scale is fixed with both the working experimental zone and the perceived exertion is measured in both of the cases i.e. before and after EWRS application. Significant changes in cardiac stress parameters and energy expenditure after application of EWRS are measured. Along with the Cardiac Frequency (CF) measurement the changes in NMC is also determined in two different experimental zones to find out the relationship between low NMC and high productivity.

Statistical Analysis

Mean and standard deviation are calculated for each set of data. Each set of data consists of spell 1 and spell 2 and before application of EWRS spell 1 variable dataset has been compared with after application of EWRS spell 1 dataset. Difference between mean values have been estimated from two different sets of data which are tested by one tail Pierson's' test (homoscedastic) with a significance level of 'p'= 0.05 and 0.0001.

Results and Observations

Results have been considered in two different application contexts one is before the application of EWRS and another is after application of EWRS. Table 1 suggests that in the case of AWHR values two different spells of before EWRS are higher than of after application AWHR. According to the workload classification of AWHR by Astrand (Astrand *et al*, 2003), it is seen that with the application of EWRS the workload is going down from very heavy workload category to heavy category. Different cardiac Cost (CC) value like NCC and RCC are also reduced significantly with the application of EWRS. Hence it is almost certain that EWRS has positively withdrawn the effect of the extra burden on mine workers.

Another significant stress estimation scale i.e. Rate of Perceived Exhaustion (RPE) are also showing positive effect on application of EWRS. Here it is clearly seen that after the application of EWRS, the perceived exhaustion scale went down to 13.91 as compared to previously 15.11.

Parameters	Before application of EWRS		After application of EWRS		t-test Homoscedastic one tail	
	Spell 1 (N=35) [Mean <u>+</u> SD]	Spell 2 (N=35) [Mean <u>+</u> SD]	Spell 1 (N=35) [Mean <u>+</u> SD]	Spell 2 (N=35) [Mean <u>+</u> SD]	Spell 1 Vs. Spell 1	Spell 2 Vs. Spell 2
PWHR(bpm)	79.3 <u>+</u> 3.35	80.71 <u>+</u> 3.82	79.5 <u>+</u> 3.4	88.35 <u>+</u> 16.93	NS	NS
AWHR(bpm)	132.07 <u>+</u> 9.03	142.16 <u>+</u> 7.91	119.01 <u>+</u> 13.84	125.64 <u>+</u> 18.25	< 0.0001	< 0.0001
NCC(bpm)	64.64 <u>+</u> 11.47	74.73 <u>+</u> 10.88	51.75 <u>+</u> 11.21	58.38 <u>+</u> 16.64	< 0.0001	< 0.0001
RCC (%)	62.89 <u>+</u> 10.4	72.76 <u>+</u> 9.53	50.54 <u>+</u> 11.04	56.91 <u>+</u> 15.41	< 0.0001	< 0.0001
Peak HR(bpm)	156.74 <u>+</u> 14.14	171.20 <u>+</u> 15.14	129.74 <u>+</u> 17.5	134.03 <u>+</u> 21.3	< 0.0001	< 0.0001
RPE	14.83 <u>+</u> 3.29	15.11 <u>+</u> 2.70	14.14 <u>+</u> 2.99	13.91 <u>+</u> 2.69	NS	< 0.05

Table 1: Effective parameters before and after application of EWRS

Application of EWRS and Energy cost

Energy Cost (EC) is an important feature of long duration work. It is believed that ECis the outcome of physical hurdles, which the body faces. In Table 2 it is seen that application of EWRS effectively reduces the Net Metabolic Cost (NMC). Here the NMC of two different spells after application of EWRS seems to be lower than that of previous tests.

Parameters	Before application of EWRS		After application of EWRS		t-test Homoscedastic one tail	
	Spell 1 (N=35) [Mean <u>+</u> SD]	Spell 2 (N=35) [Mean <u>+</u> SD]	Spell 1 (N=35) [Mean <u>+</u> SD]	Spell 2 (N=35) [Mean <u>+</u> SD]	Spell 1 Vs Spell 1	Spell 2 Vs Spell 2
EE(kcal/min)	5.70 <u>+</u> 0.58	6.28 <u>+</u> 0.68	4.77 <u>+</u> 0.52	4.98 <u>+</u> 0.70	< 0.0001	< 0.0001
NMC(kcal/min)	4.09 <u>+</u> 0.72	4.67 <u>+</u> 0.75	2.68 <u>+</u> 0.52	2.89 <u>+</u> 0.70	< 0.0001	< 0.0001

Table 2: Energy cost before and after application of EWR

The implementation of the EWRS shows a positive impact as the EC of spell 2 has been reduced significantly (<0.0001) in both cases (Table 2). The implementation of EWRS in between the working spell effectively reduces the NMC (see Figure 1). EWRS technique is applied to the same workforces and shows that NMC has gone down by 34.5% in case of spell 1 and 38.1 % in case of spell 2 respectively. The rest allowance formula proposed by Spitzer (Ayoub, 1989) is: R= (M/4-1) ×100, where, R=Resting time as a % of working time, M= Net metabolic cost (kcal/min) = (Total energy cost-resting energy cost). If NMC is considered as the rest pause variables then based on Spitzer a significant reduction in NMC is observed after EWRS as compared to before EWRS. Specifically before EWRS, NMC remains at a value that is higher than 4 kcal/min. It indicates a need of a rest pause. Reduction of NMC to below 4 kcal/min is possible with implementation of EWRS where there is no requirement of rest–pause. This will be increasing the working time of the miners and thereby with good impact on the time of Effective Manpower Shift (EMS) and productivity of the mine.

Different working spells in mines have different augmentation of work demand and the EC depends on work nature. Figures 2 and 3 show NMC differences in spell 2 and 3 respectively.

According to the American conference for governmental industrial hygiene, the EE of two different spells for EE before application of EWRS are showing the value indicating the Moderate work intensity to Heavy while after application of EWRS the same workforces are tested and is found below the range of moderate intensity (ACGIH, 2012) i.e. light to moderate working intensity (Table 2).







NMC SP2 (After WRS) (Kcal/min)

 NMC SP2 (After WRS)

 0

 0

 0

 0

 0

 0

 0

 0

 0

 0

 0

 0

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Figure 2: NMC differences in spell 1

Figure 3: NMC differences in spell 2

Schematic model of change

Figure 4 shows the schematic model of effects of EWRS.



Figure 4: Schematic model of effect of EWRS

DISCUSSION

Having considered the importance and rising productivity related problems in Indian mines along with high EC, this study on EWRS application provides basic and proven information aiming at improving productivity and EMS efficiency. Moreover, EMS efficiency improvement can also reduce production cost and increase productivity onto a certain level. In mining like other industries exposure to a bad workplace design and hostile working environment maximally affect the workers. Handling of face loading machines in underground and heavy earth moving machines in open cast mines also causes direct exposure to vibration. In this study underground tramming operation had been considered and it is seen that the physiological EC in between the spells is very high which requires a rest pause of some extend. Job profile of underground trammers in Indian mining mostly requires continuous work pattern without having a proper work rest schedule. In Indian underground coal mines, comfortable operating conditions are very difficult to maintain due to varying geo-mining conditions within the ambit of desired muscle efficiency for 8 hr shift. Working pattern is thus responsible for development of musculo-skeletal disorders with early onset of fatigue. Ergonomic intervention in the form of implementation of EWRS for one month of time has reduced the NMC significantly (p<0.0001). The mean NMC is achieved about 32.3% and 37% less with application of EWRS in mines for two different working spells. Perceived exertion for those mine workers are achieved in between Maximal Heart Rate (MHR) of 70-80%, which is considered as very high working intensity (Borg, 1982). Therefore, it is obvious that the workers are facing high job stress in mines every day. Appropriate static cardiovascular functioning is very important to carry out work in hot and humid working conditions where changes in body function directly reflect on WHR (Biswas et al, 2011). The different spell of WHR and NMC (Figures 2 and 3) are showing the augmentation in physiological job demand and work-stress regimen. Importance of WHR imposes greatly to select derived heart rate parameters like NCC, RCC and EE, which plays an important role to detect physiological job demand. In this present study it is noticed that the WHR and other derived parameters like NCC (Lablache-Combier and Ley, 1984), RCC (Chamoux et al, 1985), and EE showed a high range than the normal recommended value.

Excess demand of energy as well as high CF increases the EE and heart rate profile to minimize the increasing job demand. The body generally increases the blood flow to cope with working efficiency levels. Higher job stress, high mechanical efficiency, higher CF requires high energy output for a continuous time of 8 hours, which is practically impossible to sustain under hot and humid working condition. Short spells with adequate rest pause break i.e. EWRS application makes it smooth and regular in Indian mines which reduces not only the NMC but also the RPE. The present study indicates that application of EWRS reduces the perceived exertion from heavy-moderate category to moderate -light intensity.

Perhaps most importantly, the findings of this study have shown that Indian mining firms may benefit substantially by improving Effective Manpower Shifts (EMS) to enhance the productivity with appropriate EWRS application programs. In contrast, the benefits of applications of ergonomic intervention are clear not only by reducing pain and the EC but also with enhancing productivity. Study results suggest that ergonomic intervention in mine workplaces alone provide a sustainable productivity benefit.

CONCLUSION

The study concludes that the physiological job stress and energy cost could be minimized significantly by implementing EWRS in between different working spells in mines. The study has shown the high range of cardiac strain through NCC and RCC parameters which are beyond the normal recommended value. NCC and RCC values are high because of bad workplace design and improper work-rest scheduling in between the different working spell of a single individual. Cardiac stress parameters along with energy cost parameters go down after implementation of EWRS measures.

High CF, RPE and NMC go down to significant level, which will pose an optimistic effect on psychophysiological condition of a miner which possibly reflects a positive way to EMS and productivity of a mine. Therefore, it can be suggested that an implementation of EWRS and in depth ergonomic intervention would be very effective to minimize EC along with other EMS related problems as well as to increase miner's efficiency, effective working time and ultimately productivity of a mine.

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NEW ADVANCES IN MINE SITE GAS ANALYSIS USING GAS CHROMATOGRAPHS

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ABSTRACT: Gas analysis using mine site gas chromatographs has traditionally been restricted generally to permanent gases (i.e. helium, hydrogen, oxygen, nitrogen, methane, carbon monoxide, carbon dioxide, ethylene and ethane). Improvements in the technology now allow for additional gases such as aliphatic hydrocarbons and BTEX to be analysed. Aliphatic hydrocarbons and BTEX have implications for the detection of spontaneous combustion, currently under investigation by ACARP project C25072. This paper will detail the modifications needed to enable analysis of these gases existing mine site equipment. The analysis capabilities of the modified equipment will be determined and examples of the chromatography produced by the system provided. The stability of the samples will be analysed in terms of the existing sampling procedures and equipment utilised. The stability of the samples with regard to any recommended new sampling procedures and equipment will also be outlined.

INTRODUCTION

The micro gas Chromotoraph (GC) is the current technology used at mine sites for analysis of the general permanent gases. This technology can analyse for helium, hydrogen, oxygen, nitrogen, methane, carbon monoxide, carbon dioxide, ethylene and ethane. Analysis for acetylene is also possible, but not routinely set up for Australian coal mines.

ACARP Project C10015 identified a Volatile Organic Compounds (VOC) fingerprint for the spontaneous combustion profile of Australian coals below 100°C. A Benzene, Toluene, Ethylbenzene and Xylene (BTEX) profile was identified between the temperature ranges of 60-80°C for Bowen Basin coal. Benzene and toluene were observed at low temperatures in Upper Hunter, NSW coal. This project did not identify a C3 to C6 alkane profile – propane, butane, pentane and hexane (Clarkson *et al.*, 2007).

The technology of the time meant that BTEX required specialist sampling with a tube at set flow rates and timing, with the analysis needing to be performed in a laboratory. Challenges such as transit time to a suitable laboratory, the time taken from sampling to generation of results and the need for analysis outside normal business hours meant that this technology was not suited to a mine site application (Clarkson and Usher, 2008).

Analysis of aliphatic hydrocarbons was possible on a micro GC in 2007, however at the time the reporting limit was set at 100 ppm. Previous analysis of spontaneous combustion samples had given no results due to the high reporting limit (Clarkson *et al.*, 2007). It was not recognised that although the instrument had a high reporting limit, peaks equating to less than the reporting limit were present and could have provided a qualitative profile.

The current iteration of the micro GC has improved sensitivity, with a limit of detection of 1ppm for many components, and a larger range of columns available. This has resulted in a column being available that is capable of both BTEX analysis and C3 to C6 analysis of the aliphatic hydrocarbons. Given the knowledge gained from ACARP Project C10015, the suitability of the improved technology for a mine site application needs to be established.

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Sampling for aliphatic hydrocarbons and BTEX is established for related industries such as oil and gas, and includes the use of stainless steel cells (Fish, 2002) and Tedlar bags (Saber and cruze, 2009). Sampling with stainless steel cells is not appropriate for the coal mining industry as they are taken from high pressure pipelines. There is also the potential for some sampling media to have background levels of the target gases present (Mussato, Varisco and Tsurikova, 2009). Aluminium gas bags are the industry standard used in mining for the general permanent gases. They, along with Tedlar gas bags, need to be assessed to determine their suitability for the mining application for aliphatic hydrocarbon and BTEX analysis.

MICRO GC HARDWARE SETUP

General Permanent Gases

The micro GC typically used is a four channel chassis, which has the ability to house up to four individual channels, each containing its own column and carrier gas supply. The general permanent gases can be analysed by using three or four channels, leaving the opportunity for additional analysis with the fourth channel space. Some mine site micro GC's are setup with three channels for the analysis of the general permanent gases, the addition of a fourth channel is simply a matter of extending the carrier gas lines internally and installing the additional channel. Mine sites that currently have a four channel setup for general permanent gases will be able to replace one of the existing channels with a new channel. This work can be performed by the instrument supplier's technician, and would typically take a few hours at most to perform. The modification work would not be performed on the mine site, and so a temporary GC would need to be installed to ensure continuity of monitoring ability. Table 1 lists the general permanent gases analysed for by a mine site, and the typical setup used on a micro GC.

Table 1: Typical micro GC setup for a mine site application

Column Type	Carrier Gas	Components Typically Analysed
MS5A – Molecular Sieve	Argon	Helium, Hydrogen, Oxygen, Nitrogen, Methane
MS5A – Molecular Sieve	Helium	Methane, Carbon Monoxide
PPU or PPQ – Porous Polymer	Helium	Carbon Dioxide, Ethylene, Ethane, Acetylene*, Propane*

*Not routinely setup on a mine site GC

PPU vs PPQ channels

A PPU and PPQ column can provide propane analysis. Both columns provide similar information with a few key differences that are outlined in Table 2.

Table 2: Differences between PPU and PPQ columns (van Loon, 2012)

Variable	PPU	PPQ
Water peak	Swamps chromatography and occurs randomly	Defined peak that does not interfere with other components
Ethylene and acetylene separation	Can separate both components	Cannot separate both components, co-elute as one peak
Propane and propylene	Cannot baseline separate both components	Can separate both components

Proper maintenance of a micro GC with regular bake outs, and drying of samples before introduction to the GC minimises the issue of water peaks on a PPU column.
Ethylene is currently used as a key evacuation trigger in a mine sites spontaneous combustion Trigger Action Response Plan (TARP). Although acetylene is not typically seen in the spontaneous combustion profile of Australian coals, its contamination of a sample is possible from other sources. If inertisations equipment such as the Tomlinson Boiler and the GAG jet engine are not running efficiently, then it is possible for unburnt hydrocarbons to be generated in their output (Bell *et al.*, 1998). The use of a PPQ channel would mean that the system could generate a positive result for ethylene in such a circumstance if acetylene was present, as the column cannot separate the two.

Channels for C4 to C6 aliphatic hydrocarbon and BTEX

The target aliphatic hydrocarbons are propane, butane, pentane, hexane and their isomers. BTEX analysis targets benzene, toluene, ethylbenzene, xylene and its isomers. Several column types are available for micro GC's that can analyse these gases. Table 3 lists the column types and the gases that are able to be analysed.

Table 3: List of micro GC columns and gases able to be analysed

Column Type	Carrier Gas	Gases able to be analysed		
Alumina Oxide	Helium	Iso-butane, n-butane, iso-pentane, n-pentane, n-hexane		
Wax 52 CB	Helium	Benzene, toluene, ethylbenzene, p-xylene, m-xylene, o-xylene		
Silica 5 CB	Helium	Iso-butane, n-butane, iso-pentane, n-pentane, n-hexane Benzene, toluene, ethylbenzene, p and m-xylene, o-xylene		

The Silica column is capable of both BTEX analysis and C4 to C6 aliphatic hydrocarbon analysis. The difference between the analysis capabilities of the Wax and Silica with respect to BTEX analysis, is that the Wax can separate the para and meta xylene isomers (Vattaire and van Loon, 2011), whereas the Silica cannot (Duvekot and van Loon, 2012). ACARP project C25072 will determine if separation of these isomers is required for the mine site application. Given that the Silica is capable of both types of analysis, and there is only one free space in a four channel micro GC, it is a suitable option for incorporation into a mine site micro GC. Analysis for BTEX and the aliphatic hydrocarbons is not possible in the same run, different method parameters are required to target one or the other. BTEX analysis typically requires a much higher column temperature setting compared to the aliphatic hydrocarbons. It is therefore necessary to have two different methods setup on the instrument for the two groups of target components.

METHOD CAPABILITIES

The sensitivity of the Silica channel was challenged for both the C4 to C6 aliphatic hydrocarbons and BTEX. The PPU channel was challenged for the sensitivity to propane. This was done by determination of the LOD (limit of detection). The LOD is defined as the concentration at which five repeat injections return an RSD value of no greater than 10%. To determine the LOD for the aliphatic hydrocarbons, various mixes were generated from a certified cylinder and instrument grade nitrogen on cascading Wösthoff pumps. Table 4 shows the results of the LOD for the aliphatic hydrocarbons. A certified mix of ~2 ppm BTEX was used for the LOD, the results in Table 5 show that the %RSD is significantly better than the 10% limit. The true LOD is most likely at 1 ppm for all the BTEX components, but due to the inability to mix for BTEX at this stage, a 1ppm mix was not able to be generated.

Mean

Std Dev

%RSD

1153.4

46.15

4.00

Injection	Propane	iso-butane	n-butane	neo-pentane	iso-pentane	n-pentane	n-hexane
Conc.	5.1ppm	4.2ppm	4.0ppm	2.5ppm	1.2ppm	2.5ppm	4.0ppm
1	1622	895	1223	519	327	693	867
2	1587	861	1078	492	364	614	798
3	1515	766	1056	481	294	636	826
4	1530	743	1010	514	302	640	723
5	1455	742	1018	466	310	645	707
Mean	1542	801	1077	494	319	646	784
Std Dev	64.94	71.6	86.2	22.3	27.8	29.0	68.0
%RSD	4.21	8.93	8.00	4.50	8.69	4.50	8.67

Table 4: LOD for aliphatic hydrocarbons

aria	Benzene	Toluene	Ethylbenzene	p/m-Xylene	o-Xylene
Conc.	2.32ppm	2.16ppm	1.83ppm	3.73ppm	1.77ppm
1	1196	1086	875	1421	781
2	1114	1171	861	1506	801
3	1099	1155	885	1573	741
4	1200	1057	845	1565	831
5	1158	1152	919	1510	764

877

27.89

3.18

1515

60.84

4.02

783.6

34.48

4.40

1124.2

49.72

4.42

Table 5: LOD for BTEX

Example chromatography for low level mixes for the aliphatic hydrocarbons and BTEX components on the Silica channel are shown in Figures 1 and 2. The concentrations of aliphatic hydrocarbons in Figure 1 are: 4.2 ppm iso-butane, 4 ppm n-butane, 1ppm neo-pentane, 1.2 ppm iso-pentane, 1ppm npentane and 0.5 ppm hexane. The concentrations of BTEX in Figure 2 are the same as those found in Table 5.



Figure 1: Example chromatography for aliphatic hydrocarbons



Figure 2: Example chromatography for BTEX

It should be noted that the method used for the LOD is an analytical method, designed for use in commercial NATA certified laboratories. The instruments are capable of producing peaks that are smaller than their listed LOD. Peaks that are below the LOD can still be used in a mining application. An example of this is the use of ethylene (a key spontaneous combustion indicator used in TARPs), the current analytical LOD for ethylene on a PPU or PPQ column is typically 1 ppm. Levels of less than 1 ppm can be generated in spontaneous combustion events. The fact that the results are below 1 ppm does not mean that they are discarded, with the importance placed on the trend that is observed, and the fact that ethylene is present. The same would apply for the C3 to C6 aliphatic hydrocarbons and BTEX components, should ACARP project C25072 determine their application relevant for spontaneous combustion detection.

SAMPLING EQUIPMENT

The equipment used to take gas samples involves tubing, pumps and gas bags. Each part of the sampling equipment has the potential to cause interference in the ability to accurately analyse for aliphatic hydrocarbons and BTEX.

Gas Bags

Two types of gas bags were tested for their interaction with the aliphatic hydrocarbons and BTEX, a Tedlar gas bag and an aluminium gas bag (the commonly used gas bag for mine site gas sampling). The GC was calibrated with the known span gas using all stainless steel gas lines to eliminate any potential interactions. A Tedlar and aluminium gas bag were filled with the known standard, using all stainless steel connections from the cylinder to the bag, and immediately run on the GC to allow a comparison with the known span gas.

Figure 3 shows the results of the study for the aliphatic hydrocarbons. The difference between the span gas and Tedlar and aluminium gas bags was not significant for most gases. The exception to this was n-pentane and hexane, which showed an immediate loss of approximately 7.5% and 15% in the aluminium bag only.



Figure 3: Comparison of sample bag types for aliphatic hydrocarbons

Figure 4 shows the results of the study for the BTEX components. The difference between the span gas and Tedlar bag is not significant for benzene, toluene and ethylbenzene. The loss for all of the xylene isomers is significant, with the o-xylene being around 11%. The aluminium bags showed significant losses for all components. The smallest loss was approximately 9% for benzene, with the worst affected being ethylbenzene and the xylene isomers, all of which reported less than 50% of their expected values. This indicates that there is absorption occurring, either via the dairy tube or the internal lining of the gas bag.



Figure 4: Comparison of sample bag types for BTEX components

Sample Stability

The short term and long term stability of both the aliphatic hydrocarbons and BTEX were tested in aluminium and Tedlar gas bags. Gas bags are routinely analysed on a mine site within the same shift

or 24 hours of being sampled underground, if the mine has an on-site GC. Analysis times would be longer for mines that send gas bags externally for analysis, due to factors such as delivery times.

The results of the short term, 16 hour, study for the aliphatic hydrocarbons in an aluminium bag are found in Figure 5. All components show a decrease over the testing time. At 16 hours, iso-pentane and n-pentane had a loss of 10%, hexane was at 22% and all other components had losses of 5%.



Figure 5: Aliphatic hydrocarbons short term stability in an aluminium bag

Figure 6 shows the results of the stability study over one week. The results from the aluminium bag study show the same trend in losses as the short term study. n-Pentane and hexane exhibit the greatest decline in sample stability over the one week time frame with losses up to approximately 20%.



Figure 6: Aliphatic hydrocarbons long term stability in an aluminium bag

Figure 7 shows the results of the stability study in a Tedlar bag over seven days. This demonstrates that the aliphatic hydrocarbons are very stable in a Tedlar bag. A short term stability study was not performed, due to the results from the long term stability study.



Figure 7: Aliphatic hydrocarbon long term stability in a Tedlar gas bag

Figure 8 shows the results of the short term stability study for the BTEX components in a Tedlar and aluminium gas bag. The results from this testing show that these components are stable in a Tedlar bag over a period of 8 hours. Despite the initial losses in the aluminium bag, due to interaction with the dairy tube or internal gas bag lining, the losses for all components after six hours was 6 - 13%, except o-xylene, which was around 17%. At 24 hours, benzene had a loss of 16%, and the other components ranged from 25 - 41%.



Figure 8: BTEX short term stability study in a Tedlar bag and aluminium bag

The stability of BTEX was determined over a 13 day period in both aluminium and Tedlar gas bags, shown in Figure 9. After one day in a Tedlar bag, benzene was still stable, however significant losses were seen for the rest of the components (approximately 22% for o-xylene). Additional work needs to be performed to determine where the losses start to become significant, given that the short term eight hour study showed the BTEX components to be stable in a Tedlar gas bag. All components continue to exhibit significant losses of greater than 40% by day 13, with the exception of benzene.

The aluminium gas bag also showed significant losses over the 13 day period. After one day, the largest loss was around 18% with p/m-xylene. By day 13, all components had lost 50 - 60% of their original concentration.



Figure 9: Long term stability study for BTEX in a Tedlar bag and aluminium bag

Tubing used for sampling

A variety of tubing that could potentially be used in the sampling process, was selected to determine if any interaction with the aliphatic hydrocarbons or BTEX occurred. The tube types tested were Tygon tubing, PTFE tubing, tube bundle line and the dairy tube from an aluminium gas bag. Nitrogen was first passed through each tube to determine if any background levels of each type of component was present. This result was negative for both the aliphatic hydrocarbons and BTEX, for all tube types. A certified gas was then passed through the tube, and the exhaust analysed by allowing the GC sampling pumps to pull a sample through the tubing. This was done to determine the amount of loss of each gas. Table 6 shows the results of this testing.

Tygon tubing had minimal losses for iso-butane, neo-pentane and propane. The other aliphatic hydrocarbons showed more significant losses. BTEX had 100% losses for all components. Tube bundle line showed minimal losses for the aliphatic hydrocarbons, with the exception of n-hexane at 14%. There were significant losses for BTEX through a tube bundle line ranging from 58 – 93%. PTFE tubing showed the smallest amount of loss across both types of components. n-Hexane had

the largest loss for the aliphatic hydrocarbons at 5%. p/m-xylene was the largest loss for BTEX at 20%. The dairy tube from the aluminium bag showed significant losses for both types of components. The aliphatic hydrocarbons ranged from 13 - 73%. All of the BTEX components had losses greater than 96%.

All of the tubing required multiple injections, as the result decreased before finally stabilising. The results quoted in Table 6 are the final stabilised results. The decreasing results indicate that absorption is occurring, and reaches a critical point at which no further interaction occurs. Further injections with nitrogen through the tubing suggested that once the aliphatic hydrocarbons or BTEX are absorbed, they are not simply released by the tubing.

Component	Tygon		Tube bundle line		PTFE tubing			Aluminium Bag Dairy Tube				
	Expected Conc	Returned Conc	% Loss	Expected Conc	Returned Conc	% Loss	Expected Conc	Returned Conc	% Loss	Expected Conc	Returned Conc	% Loss
iso-butane	97.73	94.92	2.9	90.94	89.12	2.0	90.94	89.50	1.6	90.94	79.32	12.8
n-butane	96.02	91.72	4.5	90.87	88.51	2.6	90.87	89.07	2.0	90.87	72.85	19.8
neo-pentane	51.46	50.24	2.4	47.73	46.81	1.9	47.73	46.96	1.6	47.73	41.49	13.1
iso-pentane	99.81	93.20	6.6	93.17	90.70	2.7	93.17	91.27	2.0	93.17	67.12	28.0
n-pentane	98.07	85.57	12.8	93.52	88.74	5.1	93.52	90.91	2.8	93.52	54.20	42.1
n-hexane	47.01	30.33	35.5	45.92	39.50	14.0	45.92	43.60	5.1	45.92	12.39	73.0
Propane	206.0	200.9	2.5	194.8	189.9	2.6	194.8	187.4	3.8	194.8	140.2	28.0
Benzene	11.7	0	100.0	11.17	4.68	58.1	11.12	10.45	7.7	11.38	0.42	96.3
Toluene	11.0	0	100.0	10.55	1.81	82.8	10.52	9.08	11.2	10.82	0.20	98.2
Ethylbenzene	9.29	0	100.0	8.76	0.90	89.8	9.01	6.79	15.4	8.87	0.24	97.4
p/m-xylene	18.58	0	100.0	17.46	1.41	91.9	17.51	12.03	20.1	17.42	0.53	97.0
o-xylene	9.33	0	100.0	8.71	0.56	93.6	7.99	6.52	7.6	8.29	0	100.0

Table 6: Losses for aliphatic hydrocarbons and BTEX in sampling tubes

The flow rate of the GC sampling pumps is very low, compared to that through a tube bundle line for example. The low flow rate has most likely resulted in a worst case result, as there is more opportunity for absorption to occur. This will be particularly relevant for BTEX, which is very reactive. Additional work should be conducted to replicate the flow rates seen on a tube bundle system, to test for BTEX losses.

The general rule for tubing is that the more flexible it is, the more likely it will absorb BTEX and aliphatic hydrocarbons. This can be seen with the losses for the tygon tubing and dairy tube from an aluminium gas bag, versus the PTFE and tube bundle line. Sampling underground at seals should be performed when the seal is breathing out, otherwise sampling pumps that contain a flexible diaphragm have the potential to absorb the target gases.

CONCLUSION

The mine site micro GC currently used for analysis of the general permanent gases can be modified to allow for analysis of aliphatic hydrocarbons and BTEX, by installation of a Silica 5 CB channel. The sensitivity of the new channel is comparable to existing channels analysing for spontaneous combustion indicators. Should ACARP project C25072 determine the use of aliphatic hydrocarbons

and BTEX to be relevant for spontaneous combustion, the sensitivity of this channel will make low ppm level analysis achievable.

Tedlar bags are the ideal gas bag to be used when sampling for both the aliphatic hydrocarbons and BTEX, as they show the least amount of interaction as opposed to the aluminium bags. These bags, however are not what is currently used for gas sampling underground. Their use would need to involve alteration of sampling lines underground, as the inlet for a Tedlar bag is significantly smaller than that of an aluminium bag. Tedlar bags are also substantially more expensive than aluminium bags. Aluminium gas bags may be able to be used, subject to the findings of ACARP project C25072, as analysis completed within 24 hours can still obtain relevant information.

The stability of both the aliphatic hydrocarbons and BTEX would need to be considered for a mining application. Mine sites with an on-site GC could analyse the sample within hours of it being taken, minimising any stability issues. Additional work needs to be performed to determine at what point the stability of BTEX in the Tedlar bags becomes compromised, given that BTEX is stable at 8 hours, but shows significant losses at 24 hours.

Great care would need to be used when determining the type of tubing used for sampling. The use of tygon tubing for example, could lead to 100% loss of BTEX. Stainless steel is the ideal material to be used for sampling however, its use is not necessarily practical due to its rigid nature. PTFE is the most suitable out of the tubing types tested for both the aliphatic hydrocarbons and BTEX. It should be noted that the testing of the tubing involved low flow rates, and therefore gave a worst case scenario. This may be particularly relevant to tube bundle line, as sampling for spontaneous combustion indicators may be necessary through the tube bundle system if there is no access available underground.

RECOMMENDATIONS

Assuming that ACARP project C25072 shows aliphatic hydrocarbons and BTEX to be relevant in the determination of spontaneous combustion, the major recommendations for the implementation of analysis at a mine site are:

- Installation of a Silica 5CB channel into on-site mine GC's,
- The use of Tedlar bags for the analysis of aliphatic hydrocarbons and BTEX,
- Analysis as soon as possible, preferably within the same shift, for both aliphatic hydrocarbons and BTEX,
- The use of stainless steel for sampling line wherever possible,
- The use of PTFE for sampling where stainless steel cannot be used,
- Elimination of any kind of flexible plastic in the sampling line, and
- Sampling at seals when they are breathing out, to avoid potential interactions with a sampling pump.

Additional work that needs to be performed for the mine site application of the analysis of aliphatic hydrocarbons and BTEX:

- Determination of the stability of BTEX in Tedlar bags between 8 hours and 24 hours, to determine at what point losses start to occur
- Additional testing with tube bundle line, at flow rates that replicate those typically seen on a tube bundle system, to determine the losses for both aliphatic hydrocarbons and BTEX.

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UNDERGROUND COAL MINE GAS MONITORING EMERGENCY PREPARATION

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ABSTRACT: An emergency situation in an underground coal mine site is an unexpected and often dangerous event that can quickly escalate if immediate action is not undertaken. Emergency situations often occur at the worst possible time and have the potential for multiple loss of life, loss of equipment, loss of coal assets and major damage to a company's reputation. In an underground coal mine, an emergency situation involving a coal heating or small fire can quickly deteriorate into a large fire or worse a gas explosion. As the emergency situation unfolds, various activities will need to be undertaken that are not part of the normal work practices, for example gas sampling from a bore hole. The equipment required for bore hole sampling, may not be available onsite and the onsite personnel may not be familiar with its use. The aim of this paper is to provide a number of examples, from previous mine incidents to highlight some opportunities as to the emergency planning and preparation that can be implemented to improve the efficiency of the mines emergency response.

INTRODUCTION

When a high potential incident occurs in a coal mine, the situation can quickly spiral downward to cause multiple loss of life, loss of equipment, loss of coal assets and major damage to a company's reputation. Without wanting to state the obvious, a fire in a coal mine is very difficult to control as the fuel (coal) is abundant and changing the ventilation may endanger the underground mine workers.

A fire in an underground coal mine creates numerous problems –

- 1. The fire has plenty of fuel
- 2. The fire produces smoke, toxic gases and flammable gases
- 3. Gaining access to the fire may be restricted due to the underground workings
- 4. Coal is a good insulator so hot coal can readily reignite when exposed to ventilation
- 5. Using water on a hot fire can also produce flammable gases
- 6. The fire can be deep underground a long way from the surface
- 7. Gas monitoring of the fire may be restricted to monitoring at the ventilation shaft or portal.
- 8. The airflow may reverse and put flammable, fire produced gases, across the fire potentially resulting in an explosion.

It is very important to catch the fire event early so remedial actions can be taken and the fire can be put out with minimal investment in time and resources. Unfortunately, high potential events do occur and the mine has to manage the situation as it unfolds. There are various techniques that the mine can utilise in advance, to put the mine in a better position to manage a high potential incident.

LOCATION OF THE VENTILATION FANS AND PORTAL

The ventilation fans should be situated in an isolated location distant from the other mine infrastructure. Smoke from a fire will normally vent from the mine portal and/or ventilation fans. As the smoke, from an underground fire, may be toxic (high levels of CO) and potentially flammable, any infrastructure within a certain radius of the ventilation fan or portal will become an exclusion zone. Hence if the tube bundle shed, floxal $/N_2$ generators, air compressors, administration area, muster area, are in the exclusion zone, they will need to be evacuated and have the power isolated to prevent the risk of explosion. Obviously, the loss of critical infrastructure whilst trying to bring an

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emergency situation under control is an unwanted outcome. If the mine was to suffer an explosion underground then the Portal and ventilation shaft can become like a gun barrel Huntly West, Box Flat, and Moura No. 2) are examples. The control room at Huntly West looked down the portal and from an operational point of view, this was an advantage. Unfortunately, when the mine exploded, the control room (evacuated) was completely destroyed (Figure 1).



Figure 1: Huntly West portal

Having to relocate all of the mine personnel, during an emergency situation, is an unwanted layer of complexity that can be avoided by ensuring that the portal and ventilation fans are remote from the mines other infrastructure.

ROUTING OF SERVICES

It is recommended that the various services to an underground be supplied via different routes. For example, for a coal mine with a drift, it is convenient to route the power, water, compressed air, phone, Underground Intercom System (DAC), gas monitoring, down the drift. Unfortunately, if a fire was to occur in the drift, all the services in the roadway are going to be destroyed. Hence a fire, in the drift, can cause the loss of power, compressed air and phone, underground at the worst possible time. The Level 1 Emergency exercise at Cook Colliery was based on a fire at the bottom of the drift conveyor. As the conveyor was close to the surface infrastructure, many of the mines services were run down this drift including power, phone, DAC, compressed air, water, Tube bundle tubes and the realtime system. As the simulated fire gained in size, all of these services were destroyed, which impeded the mines response as the only communication with underground was via PED and the mines gas monitoring was offline. Hence, if the various services take different routes underground, i.e. via the drifts and bore holes the services damaged via a fire are limited.

Bore holes

Gas monitoring is essential to the management and resolution of emergency situations such as a fire underground. Once the gas levels underground reach the levels prescribed in legislation, the power has to be turned off to prevent the risk of an explosion. Hence the realtime monitoring will stop operation several hours, after the power is turned off and the Uninterruptible Power Supply (UPS) batteries run out of power. After the power is turned off the only monitoring available will be via the tube bundle system and bag sampling via the portal, ventilation fans and any bore holes. If the tube bundle system is monitoring locations around the fire, it would be a significant advantage. However, additional gas monitoring locations are normally required to gain a greater understanding of the fire event and additional bore holes take time to drill.

It would be an advantage to have boreholes predrilled at various strategic locations ready for gas monitoring should an emergency event occur. The boreholes do not need to be very large as they only need to have a tube to sample the underground atmosphere. These locations should be identified as areas of high risk.

The delays created by sourcing and transporting drill rigs to site in addition to actually drilling the bore holes can be quite long. The borehole delays only allow the situation to potentially deteriorate further. If the emergency monitoring bore holes were drilled as part of the development, the mine would be better placed to respond to a future emergency situation. Pike River mine is located under mountainous terrain in a national park in New Zealand. After the mine explosion on 19 November 2010, the only monitoring that could be undertaken was from the portal and ventilation shaft. Bore holes needed to be drilled to gain more information regarding the gas atmosphere underground, but unfortunately, due to the mountainous terrain, the drill rigs had to be disassembled, flown to site by helicopter and then assembled prior to the start of drilling (Figure 2).



Figure 2: Drill rig at Pike River

At Carborough Downs where in 2012, spontaneous combustion in the longwall panel required extra bore holes to be drilled. As this was approximately at the peak of the mining boom, sourcing drill rigs was more difficult and expensive.

Borehole Sampling Equipment

Once an explosion or fire has occurred, gas monitoring is essential to the response and potential rescue of trapped mine workers. The process of collecting gas samples can be potentially hazardous if the correct equipment is not available or an incorrect methodology is applied. For example, collecting a gas sample from a ventilation shaft after an explosion could expose the mine worker to toxic levels of CO and the risk of another explosion. Hence consideration needs to be given to installing an explosion proof tube for remotely monitoring the gas from the ventilation shaft. A 12.5 mm (1/2") stainless tube encased in concrete which is sampling the ventilation shaft about 5 m from the surface would allow for the collection of the explosion's afterdamp or fire's smoke when the mine is breathing out. The ventilation shaft tube would need to run to an area outside the exclusion zone and the majority of the run could be carried out in normal tube bundle tube.

In order to draw a gas sample though the tube, it would require a gas sampling kit (pump and bag filling equipment). The equipment required to draw a sample from a ventilation shaft or bore hole consists of a small diaphragm pump and various valves to fill a gas bag. Most mine sites do not have this equipment or even the parts required to build one in an emergency situation. A case in point is Pike River, where in order to sample the ventilation shaft, after the explosion, a stomach pump from an attending ambulance had to be used. Hence, having a bore hole sampling kit available would have

made the sampling of the ventilation shaft a lot faster and with reduces risk. Of course, training in the use of the Bore Hole Sampling Kit is a core requirement as is the maintenance of the equipment.

If the bore hole is not fully lined, then a tube will need to be lowered into the mine. With mines approaching 300m in depth, the process of lowering a 12.5 mm ($\frac{1}{2}$ ") tube can be quite involved as the tube gets very heavy and beyond a certain length, the tube will start to stretch hence will require a catenary cable for support. In addition, the mine environment could be very hot (melting the tube) and hence the last few meters may need to be stainless steel to survive. The last two meters will need to have a piccolo arrangement, so a cross section of gases, from the underground environment, is drawn to the surface for analysis. The process of lowering this arrangement down a bore hole is not a simple exercise and requires a tested procedure, risk assessment, training and the equipment to safely lower the tubes without risk to the mines personnel.

Tube bundle gas bag collection

One of the advantages of a tube bundle system is the ability to actually collect a sample from underground for analysis with a gas chromatograph. In order to actually collect a sample via the tube bundle system requires gas bags and personnel trained in the process. If the correct methodology for gas bag sampling is not followed, it is very easy to introduce some sample from a previous tube or contaminate the bag with fresh air. Having a ready supply of gas bags and personnel trained in the use of collecting samples would be essential to gaining a good picture of the gases underground early in the incident. A careful and considered gas chromatograph run by a highly trained operator and then interpreted by a technical expert are quickly undone if the quality of the gas bag collection is substandard.

HAZARDS CREATED DUE TO THE EMERGENCY

As an emergency situation is an unexpected event, many people are performing work outside their normal area of expertise. For example, mine workers may be charged with collecting borehole samples. Without a procedure, risk assessment or a trained supervisor, the workers may be exposed to hazards that they do not recognise. One potential hazard is the exhaust of the Bore Sampling Kit i.e. toxic and/or flammable gas. Hence the operators may be exposed to toxic/flammable gas and when the Bore Hole Sampling Kit is situated next to the generator, the hazards increase further (flammable gas being ignited by the hot exhaust of the generator.

When the first emergency bore hole was completed for Pike River, an improvised bore sampling kit was constructed and transported to the bore site. The bore sampling kit's exhaust line was not unrolled (information tag instructions not followed) and pumps exhaust was venting next to the portable generator. As the underground atmosphere was found to by approximately 95% CH₄, the potential for an explosion was present. Hence, it would be an advantage to have procedures and risk assessments for the expected tasks that would need to be undertaken if there was an emergency onsite in order to prevent personnel being exposed to risk.

Fatigue management is an area where procedures and risk assessments are very relevant as the emergency event may continue for weeks or more. The standard fatigue management system may need to be modified to take into account the emergency situation but the underlying hazards of sleep deprivation remain. In addition, it would be an advantage if the Risk Assessments for various emergency situations were competed and updated on a regular basis so these can be used as a backbone for a future real emergency. By having a backbone risk assessment, it could be updated to reflect the actual event and hopefully speed up the deployment of mines rescue, ventilation changes.

Gas monitoring infrastructure

The gas monitoring in an underground coal mine is constantly changing to suit the current mining conditions. However, the gas monitoring needs to be in front of the mining conditions and the industry is very good in being proactive on this front. However, there have been occasions where the gas monitoring has not been in place to suit the mining conditions. Hence, prior to a goaf being formed,

the tube bundle system needs to be in place and operational in order to detect the start of spontaneous combustion in the remaining crushed coal.

If the mine doesn't have a tube bundle system and there is an incident underground or the fans trip and the mine gases out, the power to the realtime system will need to be turned off, leaving the gas monitoring to the Tube Bundle system. Hence it would be of value to have tubes located in strategic locations underground to ensure that regardless of the power state underground, the gas monitoring of important locations underground can continue. Areas additional to those required by the legislation should be identified as part of the development of the mines principal hazard management plans

Explosion protected gas monitoring

The explosion at Pike River rendered the realtime gas monitoring nonoperational. With the force of the explosion, the realtime system was damaged and no longer reported the gas readings to the control room.

However, it has been found that a gas detector survived the blast in the ventilation shaft. The blast destroyed the fan housing, but the gas detector survived and when tested continued to work normally (the gas detector was badly burned but otherwise operational). Hence, there is scope to make the realtime system explosion resistance by shielding the gas detectors from the full force of an explosion down the roadway. The realtime cables, phone cables and tube bundle tubes need to installed with blast protection in mind i.e. tied at smaller regular intervals and installed where some protection is afforded. With regards to the tube bundle systems and explosions, it has been found (Brady,*et al*, 2015) that $\frac{1}{2}$ " tubes have a higher survivability that 16 mm (5/8") due to the 12.5 mm ($\frac{1}{2}$ ") tubes being more robust i.e. more difficult to kink. A mine that has an explosion protected gas monitoring system would be better placed to rescue trapped miners and recover the mine. A more robust system would also sufferer less damage from every day incidents as well.

SMOKE HAZARD MANAGEMENT

Coal smoke can be high in CO, CO2 and particulate matter which reduces air quality and can be detrimental to people with pre-existing heart or lung conditions. As the smoke from the Hazelwood mine fire forced the town of Morwell to eventually evacuate the children, elderly and people with pre-existing medical conditions, the handling of the hazards created by the smoke has been the subject of much debate. In addition, the gas sampling of a coal fire can expose the mine worker to a range of chemicals and gases not usually found in the work place including H₂S and tar. At low levels H₂S is very easily detected (smells like rotten eggs) but at higher levels the olfactory nerve is overwhelmed and after a few inhalations, the sense of smell disappears which exposes the mine worker to high levels of H₂S without their knowledge. Procedures based on risk assessments are advised prior to mine workers being in the vicinity of the mine smoke to ensure that exposure in minimised. The sampling equipment used to monitor the underground fire or explosion will collect the by-products of burning coal. BTEX (Benzene, Toluene, Ethylbenzene and Xylenes) can be found in the tar residue and these chemicals are toxic. Risk assessments on the handling and disposal of equipment contaminated by the mines exhaust would be advisable to reduce harm to the mine workers and the environment.

Efficiency of the emergency response

By having the gas sampling equipment and knowledge on how to take gas samples onsite, time and resources are saved and these can be put toward managing the incident instead. If the risk assessments and general procedures have been prepared earlier, a lot of the ground work that needs to be done prior to an activity is already partly completed so more efficient sealing, firefighting and inerting, work can be undertaken.

Data exchange

During various incidents, the sharing of gas monitoring and other data is a priority so people can work in parallel to analyse/resolve the current situation. Naturally, the various parties will all have their preferred analytical software i.e. Segas Professional, SMARTMATE and Spreadsheets.

As there are many different types of gas monitoring systems and they each have their own data format, in an emergency situation the gas monitoring data will need to be periodically (typically at the end of each shift) converted from the native data type into a common format for each of the various parties to analyse. The fast data conversion and it's uploading to the various parties is an important aspect of the emergency response. It would be a step forward, if there was a standardised gas sample format that each of the various gas monitoring systems could export their data into. A standardised gas sample format would reduce data handling errors, scaling errors and conversion errors, and speed up the generation of reports for third parties.

The Emergency Management System software used by the mine to store the events, actions, tasks, data, etc relating to the incident would be improved by having a regularly updated summary of the important information. The running summary would allow the mine worker, coming onto shift, to quickly see an overview of incident and its current status. The information and directions handed over at the end of shift, if misunderstood, can quickly lead to undesired events, hence an easily assessable running summary could reduce potential misunderstandings and well as keep everyone up to date on the incident.

CONCLUSION

High potential incidents in underground coal mine are an ever present risk in the industry. However, the mine can be better prepared to take control of the situation earlier by having some of elements of the mine designed with consideration of emergency situations, having some emergency equipment onsite and training as below -

- 1. Locate the portal and ventilation fans some distance from the mine's other surface infrastructure, so in the event of a major incident, the mine can avoid isolating surface equipment or evacuate mine workers.
- 2. Ensure that bore holes are drilled at various strategic locations ready for emergency gas monitoring.
- 3. Have the equipment and expertise on site to sample the emergency bore holes.
- 4. Ensure that the response the emergency incident doesn't expose workers to unforeseen hazards.
- 5. Ensuring the gas monitoring not only meets the requirements of "peace time" mining but also monitors other strategic locations from an emergency event perspective.
- 6. Install critical infrastructure with a view to protect it from blast damage i.e. hardened gas monitoring systems, phones, DACs, etc. monitoring system and phones may be possible.

The speed of response is critical to bringing the emergency situation under control and the deployment of mines rescue to find trapped mine workers. As mines rescue cannot be deployed until it can be assessed as safe to go underground, the ability to collect gas samples, quickly share the information and update risk assessments all help to reduce this delay.

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TELERESCUER - RECONNAISSANCE MOBILE ROBOT FOR UNDERGROUND COAL MINES

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ABSTRACT: The paper describes conception of a reconnaissance mobile robot TELERESCUER for inspecting underground coal mine areas affected by catastrophic events. The introduction describes the whole project background and the following sections deal with the design of the control system and communication between individual subsystems. The subsystems described include the motion subsystem, the sensory subsystem (temperatures, gas concentration, air flow, navigation and cameras), the subsystem for 3D map data acquisition and the communication subsystem. Mentioned also is the ATEX implementation (where the robot can safely operate in an environment with dangerous concentrations of methane).

INTRODUCTION

The goal of the project "System of the mobile robot TELERESCUER for inspecting coal mine areas affected by catastrophic events" is to develop a system for virtual teleportation (virtual immersion) of rescuers to the subterranean areas of a coal mine that have been closed due to a catastrophic event within them (Telerescuer, 2016; Timofiejczuk, *et al.* 2016; Moczulski, *et al.* 2016.). Nowadays, human rescuers inspect such areas alone. The activities of rescuers in places impacted by such disasters are extremely dangerous. Moreover, human rescuers are allowed to enter a restricted area only if the values of several critical parameters achieve acceptable levels, which often require long waiting times. To overcome these problems and improve the efficiency of operation of the human rescuers, a TeleRescuer system has been developed. The TeleRescuer system takes advantage of a special Unmanned Vehicle (UV) capable of moving within the area affected by the catastrophic event (i.e., with many obstacles, such as parts of damaged machinery and equipment, fallen rocks, damaged installations). The UV is equipped with sensors and video cameras. The requirements mentioned above have to be designed in accordance with the ATEX requirement Group I, Category M1 – "Equipment in this category is required to remain functional with an explosive atmosphere present".

STATE OF ART

There are a number of projects relating to problems of mobile robots at underground coal mines. All the projects mentioned below have one main problem in common – implementation of the ATEX in the robot design. This is probably the most important difference for these robots when compared to the "normal" field mobile robots. One of the projects, a Chinese mobile robot, shown in Figure 1 (Gao Junyao, *et al*, 2009) – is a robot with six tracks, which weighs is 65 kg, travels at 3.2 km/h maximum speed, has about 4 hours of working time (about 2 hours if moving continuously), can climb a slope of 30 degrees and can communication over a distance of 1 km. Additionally, it can carry 5 kg of food or medical supplies. The robot shown in Figure 1 consists of a mechanical vehicle, driving system, control system, communication system, sensor system, storage batteries and remote control system.

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Figure 1: Chinese coal mine rescue robot

Another example of a mobile robot designed for usage in coal mines is the Mine Rescue Robot (MINBOT) described by (Wang, *et al*, 2014). Its second generation – MINBOT-II – is developed based on the experiences learnt from the applications and experiments of the first generation (MINBOT-I) shown in Figure 2.



Figure 2: Minibot-I robotic platform

The mobile robot NUMBAT shown in Figure 3 is a mine reconnaissance robot designed in the 1990s by the Australian Commonwealth Scientific and Industrial Research Organization. The Numbat is an eight-wheeled mobile platform with an on-board gas analysis package to provide information on the environmental conditions within the mine (Jonathon, *et al*, 1998).



Figure 3: Robot Numbat

The mobile robots for coal mines described above have some disadvantages like their large size, teleoperation only (no autonomy), no ability to create a 3D map of the surroundings and most probably, problems with meeting the actual ATEX requirements. Other serious problems include: communication distance is shorter than required, ability to overcome obstacles is low, autonomous movement ability is weak or non-existent. Some tracked robots are not suitable for crossing rough surfaces caused by an explosion in a coal mine. Thus the practical robot for detection and rescue in

coal mine should have the abilities of movement on rough surface with obstacles, in smoke-filled and dusty environments, must have the explosion-proof design (Chinese standards for these conditions are less stringent when compared to European standards) and a waterproof design, many sensors, wireless and safe communication (optimally with a backup line), a teleoperation system and other subsystems like a 2D or 3D laser scanner for map building, autonomous behaviour, etc. Other information about similar projects are mentioned e.g. in Kasprzyczak, *et al*, 2012; Kasprzyczak, *et al*, 2016.

SPECIFIC PROJECT OBJECTIVES

The identification of needs has been carried out in close collaboration with the Central Mining Rescue Station (CMRS) (Timofiejczuk, *et al*, 2015). First, the rescuers filled out special questionnaires. Additionally, interviews with experienced rescuers and discussions with the higher engineering personnel of CMRS have been performed. Results of those activities have been summarized and reported (Moczulski, *et al* 2014).

The platform should have the ability to pass obstacles such as conveyor structures, conveyor drives, excavation protection structures and their intersections, hydraulic or wood racks, railroad tracks, turnouts, loading ramps, winches, transformers, switchgear or single switches, pumps, hoses, drainage, metal sheets, elements of concrete, construction machines and their fixing – beam, struts, chains, wire ropes, tubes, pipes, cables, ventilation fans and chutes. In mine roadways there can be present bulky materials (over 25 cm) left (abandoned) during transport, such as mining carts, platform, locomotives, mine roof sections, components, parts, roadway support arches, concrete lining, mesh lining, bales of ventilation cloth, metal and wooden racks, structural wood in the form of timbers, planks, beams, metal crates and boxes used for transporting spare parts.

It is required to carry out measurements, transmission, visualization and recording of temperature and humidity and temperature of selected elements of the robot body in a continuous way. Exceeding the temperature threshold shall be indicated. The operator should be able to easily and quickly program the contents of the measurement cycle. Registration of the results should include time stamps. Placement of some sensors (CH_4 , O_2) on a vertically retractable telescopic mast (independent of the arm) – enables rising sensors up to 3 m. The required measurements of environmental parameters and composition of mine atmosphere are:

- Methane (CH₄). Place of measurement: under the roof. Range: 0 to 5 % and 5 to 100 % vol.
- Carbon monoxide (CO). Place of measurement: at face level. Range: 0 to 10000 ppm.
- Carbon dioxide (CO₂). Place of measurement: near the floor. Range: 0 to 5 % vol.
- Oxygen (O₂). Place of measurement: usually at face level. Range: 0 to 25 % vol.
- Air flow (velocity and amount of air flow). Place of measurement: various methods but usually at the whole cross section of the excavation. Measurement range: 0.2 to 20 m/s.
- Temperature and relative humidity. Place of measurement: usually at face level in place of free flow of air. Temperature range: -20 to +60°C. Relative humidity range: 0 to 100 % RH.

ACTUAL STAGE OF DEVELOPMENT

The TeleRescuer robot shown in Figure 4 consists of the main body with tracked arms (motors, motor controllers, batteries and control system are encapsulated in a flameproof housing), a sensor arm with a camera head (three degrees of freedom), a 3D laser scanner unit and a mote deploying subsystem (motes are small Wi-Fi repeater modules with their own independent power units).

The most important part of the mobile robot is the sensor head on the sensor arm. The sensor head is a cylindrical module with a flameproof enclosure. It is energy independent with its own batteries. Communication with this module is done only through optical fibres (Ethernet). The sensor head

consists of five cameras (two for 3D vision, one with a wide field of view for site surveys, one for rear vision and one thermal camera).



Figure 4: TeleRescuer body (view from back): 3D model – left and its visualization – right (by www.i3D.pl)

This camera subsystem also contains remote lighting based on LED lights, various gas sensors and an inertial measurement unit (IMU Module). The control system for the sensor head is an embedded CPU board. Elevation of the sensor head, its rotation and lifting of the methane arm is realized with only one EC motor with four electromagnetic brakes – selection of the type of movement (lifting cylinder, rotating cylinder and lifting methane arm) is done by these four brakes.

The 3D laser scanner shown in Figure 5 is based on a 2D laser scanner Sick LMS111 (it is designed for black surfaces). Because the laser beam measures only in a single plane, it is necessary to rotate the laser scanner to cover the whole space. The result of this measuring is a set of points usually called a point cloud (Olivka, *et al* 2016a; Olivka, *et al* 2016b).



Figure 5: 3D laser scanner – 3D model and second prototype

The design of the first generation of the control system for this robot is in detail described in (Kot, *et al*, 2014). The control system of the mobile robot TeleRescuer (MCS – Main Control Sytem) is based on an industrial PC board with a small footprint – format Nanol TX (NANO-BT-i1-N28071-R11 from iEi company) with a mSata Solid State Disk and an IRIS module for remote management. This board is encapsulated in a special box with the modules for the optical communication and Inertial Measurement Unit (IMU) being placed in a flame-proof enclosure "d" – common with the robot chassis. In the chassis is also embedded a methane sensor. If this sensor detects the presence of higher amounts of methane inside the robot body it sequence first disables all the motor controllers'

functionality (thus minimizing current consumption) and subsequently disconnects the power subsystem. This way, the double explosion-proof safety required for Group I - M1 category is met.

Eight EC motors (4 for the track motion of each tracked arm and 4 for the rotation of these arms) provide movement of the mobile robot. These motors are driven by four dual-channels RoboteQ motor controllers connected to the MCS by the CAN bus. Each driver has an independent battery pack with common grounding.

There are two independent subsystems for communication with the operation station. The first is optical communication by optical fibre (solution n by Sedi-Ati Company). The secondary (backup) communication channel is via wireless communication with a lower speed than through the optical fibre, with a pack of releasable wireless repeater stations (motes). The subsystem are shown in Figure 6.



Figure 6: Robot subsystems and their connections

MIGRATING TO ROS

The main change in the TeleRescuer control system is in the field of the operation system. The original draft (Novák, *et al* 2014) was based on Microsoft Windows for both components (robot and operator controller) but new requirements for the control system (especially the need for programming of autonomous behaviour) forced to the migration to Robotic Operation System (ROS).

ROS is an open-source, meta-operating system for robotics systems. It provides the services expected from an operating system, including hardware abstraction, low-level device control, implementation of commonly-used functionality, message-passing between processes, and package management. It also provides tools and libraries for obtaining, building, writing, and running code across multiple computers. (In reality ROS is not a real operating system, but an extension of Linux. ROS represents a verified framework for robot design with a wide range of supported devices and libraries with many implemented algorithms. It is worth mentioning that Octomaps are an inseparable

part of ROS. The usage of Octomaps and ROS is currently presented in many projects and robotic competitions.)

Figure 6 describes the architecture of the embedded (on-board) main control system. The system is logically divided into parts (ROS nodes). Nodes marked by green (*Motion, Sensors etc.*) are responsible for the communication with the hardware components of the robot (motor controllers, sensors etc.). Orange nodes are responsible for the autonomous behaviour of the mobile robot (*Autonomy Node, Odometry node, 3D Mapping Node*) – (Olivka, *et al* 2016a; Olivka, *et al* 2016b).

The blue node (*Operator Bridge*) is responsible for communication with the external operator control panel and software simulator. This software component provides translation of the internal ROS communication between the individual nodes to communication datagrams designed at the beginning of this project. Integrating ROS into the original design for the control system brought many new challenges. Because design of some components of the control system had been already almost completed (virtual simulation system and operator visual user interface), the originally designed communication protocol had to be preserved. A special custom software module (Operator Bridge) translating TCP/IP telegrams into ROS Topics/Services was created and to ensure the command-answer principle, which is problematic with Topics (unidirectional stream of data without replies). This may provide an additional source of time lag in the communications.



Figure 7: Diagram of TeleRescuer control system – SW model

ROS COMMUNICATION LATENCY TESTING

The previous version of software was designed with TPC/IP communication using custom telegrams with a strict command-answer principle. After moving to ROS, the system had to be adapted to the ROS communication protocol. The two basic standard ways of communication between ROS nodes are:

• **Topics** – one-way stream of messages established between a publisher and subscriber(s). Any node can register as a subscriber to any topic published by another node and will automatically start getting all the messages on the topic. • Services – pairs of messages (request and reply) exchanged between two nodes. A client node calls a service provided by a server node by sending the request message, the server node processes the request and answers with a reply.

The command-answer principle would suggest the usage of ROS Services. However, cleaner from the ROS point of view would be to use of Topics for periodic commands or data feedback. For better understanding of the internal implementation of Topics and Services, a series of performance tests was performed. The goal was to to optimize data the exchange between the subsystems (implemented as nodes) of the control system in order to get as low a latency as possible.

Tests concerning Topics used one or multiple publisher nodes and one of the multiple subscriber nodes. The message contained the actual timestamp (64bit integer with resolution of nanoseconds) and monitored was the latency between publishing and receiving of the message. Presented times are averaged and rounded. Figure 8 shows configurations of NODEs for individual ROS Topics latency tests, and Table 1 shows average latency results.



Figure 8: Configurations of NODEs for individual ROS TOPIC latency tests

Table 1: Averaged latency results for ROS Topics (multiple values in a single test correspond to individual receiving nodes)

Test	Description	Latency [ms]
A	1 node publishing 1 topic to 1 subscriber.	0.5
В	1 node publishing 2 topics, 1 of them is subscribed to.	0.5
С	1 node publishing 2 topics to 2 subscribers	0.5, 0.65
D	1 node publishing 1 topic to 2 subscribers.	0.5, 0.6
E	1 node publishing 2 topics to 1 subscriber.	0.55, 0.6
F	2 nodes publishing 2 different topics to 2 subscribers.	0.5, 0.65
G	1 node publishing 1 topic to 3 subscribers.	0.5, 2.0, 2.8

The tests show for example that a topic which is published but not received by any node is internally not processed by the ROS communication system (test B versus test A) and that there is an interesting leap in latency times when a topic is received by 3 nodes compared to 1 or 2 (test G versus tests A and D). The other numbers are quite even and show that the tested combinations are not significantly different.

The main test concerning Services was performed to compare the latency of a Service and a Topic. Measured was the average delay between sending a Service request and receiving it on the server node and the total time between sending a Service request and receiving a reply to it. (The server node replied as quickly as possible and was not doing any additional work, which does not fully correspond to real uses – the test shows the theoretical minimal possible delay.)

ROS offers also a *Persistent service*, where a permanent connection between a client and server node is established, which is useful if a lot of Service calls are expected between two particular nodes. This option was tested the same way as standard Services. Table 2 provides average latency results for ROS services.

Service	Measured value	Latency [ms]
Standard	Client – server (request only)	2.3
	Client – server – client (request + reply)	7.8
Dereistent	Client – server (request only)	0.24
Persistent	Client – server – client (request + reply)	5.1

Table 2: Averaged latency results for ROS services

The results show that the standard Service is much slower than a topic and thus should be avoided in latency-critical situations. The persistent Service is extremely fast in the first phase (request), even faster than topics; the total latency including reply is however much higher again.

CONCLUSION

TeleRescuer is an international project managed by a consortium composed of the Silesian University of Technology (Poland), the VSB – Technical University of Ostrava (Czech Republic), the Universidad Carlos III de Madrid (Spain), COPEX (Poland), SIMMERSION GMBH (Austria) and SKYTECH RESEARCH (Poland).

This paper describes design of a control system for the mobile robot TeleRescuer. For the easy implementation of the autonomous behaviour of the mobile robot and simple teamwork across international consortium, it was decided to migrate to the operating system ROS, which is designed for usage in mobile robotics to simplify R and D software, especially in larger teams. Autonomy is crucial in cases of loss of communication with the operator, because it will be able to drive the robot back and try to re-acquire the signal.

Structure of the control system was completely re-designed for ROS. For a better understanding of the internal implementation of the ROS communication a series of performance tests were performed, in order to be able to better optimize the design of the individual subsystems and the communication between them. It was decided to change some aspects of the previous control system, for example the strict command-answer principle used in communication telegrams (TCP/IP) was replaced in most cases with one-directional streams of data (ROS Topics).

The robot can be controlled remotely via optical fibre (primary line) or wirelessly (backup line). Specialized software (Moczulski, *et al*, 2014) is used for virtual teleportation. More detailed information about TeleRescuer subsystems of 3D map building and 3D map visualization are described in the Midterm Report (Timofiejczuk*et et al*, 2016; Kot, *et al*, 2016a; Olivka, *et al*, 2016a; Moczulski, *et al*, 2016).

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HOW TO PLAN A SAFE AND SUCCESSFUL PERMEABILITY TEST PROGRAM IN COAL SEAMS

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ABSTRACT: While discussing coal permeability testing, it is often asked what constitutes a successful test and why. An understanding of these concepts will assist the development of an optimum test program that uses appropriate test equipment and procedures. This approach will provide reliable results, which in turn will improve the design, exploitation and economic recovery of gas and coal resources.

The main purpose of this paper is to explain what to do and how to do it safely to complete a successful and cost effective testing program. In this paper, test guidelines are provided which are based on more than 60 years combined experience and over 3,000 tests. Topics discussed are:

- 1. Understanding and discussing the test objectives between the operator, testing provider, gas drainage and ventilation design engineers to ensure all requirements are known, considered and met.
- Selection and use of the appropriate testing procedures Drill Stem Test (DST), Injection Falloff Test (IFT), Step Rate Test (SRT) and vertical temperature measurements.
- 3. Selection of testing equipment and related information including packer type, testing strings, pressure recorders, packer spacing based on available coal seam data, borehole logs, reference level, wellbore location maps and testing results in nearby boreholes including gas content and regional pressures if known.
- 4. Well conditioning practices to clean out debris and mud cake to ensure that the test results reflect the in-situ coal properties.
- 5. Appropriate relaxation time after the completion of drilling operations to ensure the coal is properly relaxed to reflect as close as possible the in-situ conditions.
- 6. Full integration of test results to define the coal characteristics and in-situ stress, presenting the results ready for analysis. Pressure, temperature and permeability should be reported with reference to the sea level depth to evaluate the structural effects.

Setting Test Objectives

The proximity of a mining operation affects the coal characteristics in a variety of ways. Typically, a reduction in seam reservoir pressure could affect the DST, IFT and SRT procedures. Therefore, it is important that all parties be engaged in the planning of any program at an early stage. This includes the operator, testing provider, and gas drainage and ventilation design engineers. Experience shows that detailed planning based on analysis of the available data results in less costly delays and program modifications due to site specific conditions. Successful testing is about informed planning from the outset and by having an onsite multi-procedural testing capability. Well Testing is all about getting out there, testing the wellbore and "seeing" a representative amount of coal without exceeding the individual test parameters.

Test Procedure Selection

Each test procedure: DST, IFT and SRT provide permeability and pressure; however some procedures are suited to produce more reliable results for particular coal conditions. For example, a

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DST should not be undertaken in near or fully gas saturated seams as the simultaneous production of gas and water (two phase flow) will not provide the absolute permeability we are trying to measure. In this case the IFT procedure is most useful since only water is injected into the coal. The IFT test can be compromised in low coal stress environments where even moderate injection pressures "jack the cleats open" or fracture the coal seam. This will falsely indicate high permeability. In this case the DST or the SRT should provide more reliable results. The combined SRT and fall off procedure provides the fracture opening and closure pressures as well as the coal minimum stress. Permeability is derived from the Diagnostic Fracture Injection Test analysis (DFIT). The coal stress data is presented with the permeability results derived from DST, IFT and DFIT to help correlate the relationship between stress and permeability.

Note that the coal seam pressure is usually unknown in exploration areas. For this reason, the test operator should have flexibility in the field to decide the best test to run based on the conditions encountered. If the three procedures are run in a seam and the results coincide, this indicates "high confidence". However, this is not usually the case because each procedure is affected differently by the stress and the direction of the flow. For example, in a DST as the fluid flows from the coal to the wellbore the coal relaxes because of the reduced pressure. In comparison, in the IFT the flow is from the wellbore to the formation and the coal is stressed by the injection.

When the test results produce two out of three permeability readings in close proximity these are selected as the most representative data. It is considered that the comparatively slight increase in time on site required to run the three procedures is readily justified as it assists mine ventilation and gas drainage engineers to produce designs that are more accurate.

Establishing the reservoir pressure: This parameter is vital to confirm that the seam is stable and ready for testing. It is well worth the time spent and particularly applies to coals near mining operations. For consistency, all tests need to start and finish as closely as possible to the seam reservoir pressure. To help achieve the testing objectives the operator should also have capability to assess testing on site so they can respond whilst still there.

DST cushion size: There have been repeated attempts to estimate DST cushion size rules of thumb, which are a very important part of the testing strategy. Since it is necessary to induce a controlled flow from a coal seam, it is important to size a water cushion that is small enough to allow water to flow but not the gas! Three parameters are required to calculate the cushion accurately:

- a. Reservoir pressure
- b. Langmuir isotherm coefficients
- c. Gas content in the coal seam

Assuming the size of the water cushion is risky, and may result in the test failing. Why? Because before conducting the test it is important to be reasonably sure about the value of the reservoir pressure. This is not hard. By simply setting the straddle packer across the target formation this pressure can be measured and the cushion size to achieve a successful DST can be determined.

If a short cut is taken and a reservoir pressure from the normal hydrostatic is assumed water cushion that is too large may be used. Therefore the DST cushion may be injected into a low pressure coal seam, completing an IFT to the surprise of the Tester and the Company man. This is shown graphically in Figure 1. Both the pre flow and main flow have negative slopes indicating water is not produced from the coal. In this case the water cushion is being injected into the coal. At the end of each flow, the pressure falls very rapidly instead of building up. This is another consequence of starting a test not knowing the basic pressure conditions, and using the incorrect water cushion.



Figure 1: Results of a planned shallow CSG DST that is actually an IFT

Maximum injection pressure: To produce a reliable permeability estimate, the injection pressure must not exceed the coal stress or fracture will result. Exceeding the coal stress generates high injection rates which result in erroneously high permeability. The injection rate must be set to ensure a minimal pressure increase over the duration of the injection period without exceeding the coal stress. In some low stress environments, simply applying the full hydrostatic head pressure to the coal will exceed the coal stress. An obvious sign of this is unusually high injection rates. A good indicator is assessing the permeability derived from the DST test. If this shows a good history match, then there is a reasonable probability that the IFT permeability is correct. If the IFT permeability is too high compared to the DST results, then there is a probability that the stress was exceeded and the IFT test is invalid.

A SRT consists of multiple injection rate increases of equal time durations. It is undertaken at the end of the test sequence. The DST should always be run first since the minor drawdown used will not damage the wellbore. The IFT is run as the second test without moving the packers so the same test interval is used. The well is tested using this rationale on the way down. The SRT is run on the way up. Even when there is some minor fracturing close to the wellbore this does not create problems when moving the tool upwards after each seam is tested. The SRT will induce pressure increases into the formation by increasing the injection rates as shown in Figure 2, taking the formation to breaking point, defining the Fracture Extension Pressure. Even though the SRT is completed in 40 minutes or so, it provides very useful information to avoid exceeding the fracture pressures during the subsequent IFT.

A regression analysis using the Nolte Smith G function applied to the pressure fall off data provides the Closing Pressure as shown in Figure 3. This then provides the Effective Stress which is defined as follows:

Effective Stress = Fracture Closing Pressure – Reservoir Pressure

For this reason, the Effective Stress represents the net pressure (stress) confining the coal seam, which is also called the coal confining stress.



Figure 2: Step rate test of coal seam, with estimation of Fracture Extension Pressure



Figure 3: Coal closing pressure determination

Test equipment selection

The packers: For a long time DST and IFT tests were conducted with mechanical packers which were set with string weight. In open hole testing sometimes this weight on the string caused the packer to slide down when the formation was not competent or was fragile, causing not only an invalid test but also borehole instability that could generate tool trapping. If the conditions are favorable, the mechanical packers work. Since the introduction of inflatable packers, which are more reliable and easy to operate, test operators prefer their use.

Straddle inflatable packers are now very reliable particularly those using water as the inflation fluid. Experience operators do not recommend packer inflation using high pressure gas since this increases the risk of well site accidents. It may also create wellbore control issues if the gas is released down hole due to packer failure.

Test tool design: Most test holes intersect multiple coal seams at various depths and pressures. When the straddle packer inflates to isolate a specific coal seam it also isolates the section below the test tool from that above. If a seam below the tool has low pressure (a pressure sink zone), any gas generated in the hole would migrate to this sink zone during the test duration, which is typically 12 - 36 hrs. The formation of a gas pocket below the tool, when the packers deflate can cause a well "kick"

as this free gas now rises to the surface pushing water out of the hole as it comes up. Typically, the current pressure of all the seams intersected by the well is unknown at test time, therefore, it is difficult to prevent this problem. To mitigate this effect, the tool design incorporates a fluid connection path to equalize the pressure above and below the tool. This is an important safety feature that has proved successful for many years.

Testing strings: Typically, permeability service providers utilize two types of strings: - Wireline Drill Rods and dedicated Test Strings.

Wireline drill rods: When used for testing purposes, these can leak at the tool joints - even new rods tend to exhibit some leakage and old rods are prone to more severe leakage due to fatigue. Permeability estimates from the DST and IFT are directly related to the accuracy of flow rate measurements. Hence, a leaking string will increase the apparent permeability of the coal. The water pumped from surface is assumed to be totally injected into the target coal when in reality a fraction of that is leakage. Furthermore, due to the small wall thickness of the drill rods, the threads do not have a perfect metal-to-metal seal. The use of liquid Teflon or other materials to prevent leaks is problematic as the deformation and stress induced onto the threads by these materials tends to shorten the life of the test string. Therefore, there is a high probability that a wireline drill rod, which is often used by the drillers, will have some leakage compromising the test results. In particular, when permeabilities are small, even a very small leak will create a significant error in permeability. Have you ever heard the comments "the coal permeability is low and the permeability estimates are all over the place"? When considering the methodology used, leaking strings may have contributed to that problem and this poses a question. Is the scatter real, or a result of adopting less than best practice? This is a dilemma for the end user given what is potentially at stake. Therefore, if wireline drill rods must be used for permeability testing, to confirm the validity of each result, it is recommended to test and record for rod leakage before and after each test is conducted.

Dedicated test strings: Leak proof test strings provided by test operators have a major advantage. They are designed solely for the purpose of testing and ensure that 100% of the injected or produced fluids are accounted correctly. Figure 4 provides an example of how a leakage error affects the water rate and the permeability estimate in a coal seam. This chart simulates a coal seam where the only variable is permeability. Note that a rate increase from 100 L/d to 200 L/d corresponds to a permeability increase from 2 md to 4 md. This is a slope of 50 L/d per md or about 2 L/h per md. Therefore, if the injection rate has an error of say 2 L/h the permeability will be overestimated by 1 md. Assuming the true permeability was 10 md and that because of the leakage the permeability will be computed 11 md, with 10% error. Conversely, if the true permeability was 1 md, the permeability will be calculated at 2 md with a 100% error.

Due to this effect, leaks through either a test string, surface lines or in the packer's bypass will result in an apparent increase in permeability. This problem becomes more serious as permeability decreases or the injection pressure rises. A professional test operator will always detect and solve equipment and packer leaks aiming to provide quality results.

Pressure recording: Technology changes in the last 30 years have created a revolution in well testing. Operators have moved from mechanical pressure gauges that used metallic pressure charts that were loaded into a chart reader with micrometers, so that pressure readings could be made every few minutes. This was time consuming and potentially inaccurate. Electronic memory gauges record downhole pressures and temperature. However the test string had to be brought back to surface to recover the gauge, and download the data for analysis. Currently Surface Read Out systems that provide real time pressure and temperature data at the surface as the test proceeds are used.



Figure 4: Possible errors due to a HQ test string leakage of 2 L/h

This allows the operator to "drive" the test through its stages ensuring "best practice test procedures" and allows optimization of test duration and results. With this approach, the data quality has improved significantly whilst also reducing costs by eliminating unproductive time.

Temperature profile from DST data: The Surface Read Out system allows the pressure and temperature data to be seen at real time as the test develops ensuring test optimization and minimum duration. However, we do not always use all the information available. Although pressure gauges record pressure and temperature, most of the time only pressure data is analysed. Traditionally, geological logs are used to record the coal seam depth and hole temperature immediately after drilling. However the recorded temperature is lower than the actual reservoir temperature due to the cooling effect of the drilling fluids. The main advantage of the DST temperature is that the test is run a few days after the well is drilled after the reservoir has had sufficient time to recover to its true temperature. These readings are far more representative than those derived from the geological logs because the DST produces water from each isolated coal seam.

Very accurate DST temperature profiles for a number of projects have been generated, confirming the reservoir temperatures at which the LANGMUIR ISOTHERMS should be prepared, Figure 5. Temperature data also helps pinpoint potential cross flows between coal layers. This is of interest to the mine design or development engineers. Adjusting the depth scale to subsea depth, allows for a meaningful correlation of temperatures across the field/mine area, identifying cross flows between seams. Underestimating the reservoir temperature implies over estimating gas contents which may considerably increase the mine development costs.



The message is: to avoid costly mistakes, use the temperature from the DST's that are run a week or two after drilling the wells.



Well control

Although Well Control is a specialized topic beyond the scope of this paper, some points need to be observed. Well Control aims to keep the well and crew safe during the test and prevent uncontrolled gas flow into the hole by keeping the hole full of fluid at all times. This is the best insurance and the basis of well control principles. The hydrostatic pressure must be greater than the seam desorption pressure to ensure no gas flows into the wellbore. To avoid a potential gas kick, it is strongly recommended not to deflate the packers until both the test rods and annulus are full of fluid, and the coal seam is essentially at reservoir pressure. This ensures that there is no potential gas kick from the seam. Once the packers are deflated control could be lost since it takes valuable time to re-inflate. Pulling the test tools out of the hole (POOH) causes swabbing and potentially induces gas flow. The operator needs to POOH slowly while keeping the hole annulus full with fluid to prevent gas kicks. This is mandatory. Moreover, remember that test tools/rods have volume and as removed from the hole, the hole water level will drop!

Be wary of holes that suddenly appear to be making water (like artesian wells). If the water is lifting and flowing from the hole, there is a reason. This could be gas release from a seam pushing the water out.

Well conditioning

During drilling operations the mud viscosity is increased with additives to help keep the hole clean by lifting the cuttings to the surface. It is always recommended to have some extra rat hole below the lower coal seam to allow for the accumulation of fill, leaving sufficient room for the test string and tool to operate properly. After reaching the total depth, the hole is circulated clean and logged in preparation for testing. Quality testing requires that the wellbore be circulated clean with water before the test, to remove viscous drill mud and cuttings. When estimating the coal permeability by measuring the water flow through the coal, contaminated fluid will affect the results. If an IFT is conducted and viscous mud is wrongly injected into the coal instead of clean water, then the estimated permeability is going to be significantly lower. As a result the coal could be dismissed by the miner or the coal seam gas operator.

For example Figure 6 shows that the injection rate in a 2 md coal is inversely proportional to the fluid viscosity. For the same injection pressure, this 2 md coal could take either water at 230 Lpd or 2.95 cp mud at 66 Lpd.





However if the test injects mud instead of water and the viscosities are not used correctly, the estimated permeability will be wrong as shown in Table 1.

Injected Fluid	Rate	Viscosity (Cp) @	Estimated Permeability	Comments
Injected Fluid	(Lpd)	30 Deg C	(md)	
Water	230	0.84	2.00	Correct
Mud	66	2.95	2.00	Correct
Assuming Water		0.94	0.57	In connect Donne
instead of Mud	00	0.84	0.57	mcorrect Perm

 Table 1 – Example showing relationship between rates, viscosity and permeability

If the well was not conditioned properly and the mud was not fully replaced by clean water, we could still salvage the IFT by recalculating the permeability using the drill mud viscosity records that are traditionally measured and recorded at the drill rig using the Marsh Funnel.

Post drilling seam relaxation time

In the majority of cases, the coal seam reservoir pressure is observed to be lower than the full hydrostatic head pressure. Drilling through such an environment exposes the seam to this full hydrostatic pressure, essentially resulting in an "injection test" because of the "positive" pressure increment exposure. In our view, it is important to let the water level in the hole stabilize upon drilling completion for a few days prior to testing to let the seams equilibrate to a stable condition. This needs to be coordinated with the Operator to avoid potential hole stability issues.

Integration and presentation of results

Step rate test and DFIT analysis - Data integration

The use of the DFIT allows the permeability results across a site to be placed in context, comparing the DFIT, DST and IFT permeability results against the minimum stress. This is very important since it provides an opportunity to critically review and integrate the results of various testing campaigns in a particular area where stress and faulting occur. Figure 7 presents an example.



Figure 7: An example of integration of field permeability and minimum stress

In low permeability conditions both the DST and IFT may have limited radii of investigation and both can be materially influenced by near well bore damage (high skin for example). This could result in a situation where the DST and IFT potentially underestimate the coal permeability. The recent

development in DFIT analyses allows permeability estimation based on fluid diffusion in the fractures within the coal. Fracturing the coal allows the test fluid "to get out there" and see clean undamaged coal.

Finally, the pressure, temperature and permeability results estimated at specific depths need to be reported at a sea level depth so as to account for the topography/changes in the area. This is computed using the wellbore elevation and the well logs depths.

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OUTBURST THRESHOLDS – MISCONCEPTIONS, CRITICISMS AND CONTEXT

Mark Blanch¹

ABSTRACT: This paper describes the origins of outburst thresholds and considers the role gas content and gas desorption rate measurements play in outburst management systems employed in the Australian underground coal industry. It considers and provides context around criticism of simplicity, conservatism and the scientific basis of gas content thresholds and desorption rate indices. The validity of increasing thresholds based on reduced advance rates is questioned as is our ability to predict outbursts and the magnitude of those events.

BACKGROUND

Up to 1994 there had been around 800 outburst reported across the Australian underground coal industry (Harvey, 2002), most of those occurring in the Bulli seam mines. There have been a total of 21 deaths associated with outburst, the most recent being the triple fatality at South Bulli Colliery on the 25th July 1991 followed by the single fatality at West Cliff Colliery on 25 January 1994. Following the South Bulli fatalities the industry adopted a risk management approach (Harvey, 2002), utilising prediction and prevention techniques with protection as a fall back in the event that an outburst did occur. In May of 1994 the NSW mines inspectorate issued a notice pursuant to Section 63 of the Coal Mines Regulation Act (CMRA) which placed restrictions, prohibitions and requirements on all coal mines operating in the Bulli seam:

- a. The gas content and composition ahead of the face was to be known (measured according to AS3980)
- b. Structure identification ahead of development roadways was required. Where structure was identified mining was only to be carried out under outburst mining procedures
- c. Normal mining was only to be carried out where:
- d. No structure had been identified and
- e. Where the total gas content was measured to be less than 9 m³/t CH₄ and 5 m³/t CO₂
- f. Mining in gas contents higher than those thresholds was only permitted under full outburst procedures, or remote mining
- g. Where gas content was measured to be greater than 12 m³/t CH₄ or 8 m³/t CO₂, only remote mining was allowed
- h. General body CO₂readings were required at the working face of development panels every 2 hrs
- i. Training was required across the underground workforce in all aspects of outburst hazards (signs, dangers, rescue and escape)
- j. Refresher training was required every 6 months
- k. First response rescue and escape equipment was required in each panel
- I. The need to comply with design and operational requirements for machine operator protection as stipulated by the chief inspector of mines.

In the subsequent years Outburst Management Plans (OMP) were refined and systematic predrainage programs were implemented at all mines using directional drilling techniques. Integral to the success of the OMPs has been the Authority to Mine (ATM) procedure, outburst management committees and clear definition of roles and responsibilities. During the mid to late 90s the Bulli seam mine OMPs were adopted across the Hunter Valley and Bowen Basin mines where seam gas was

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identified as a hazard. Outburst thresholds for non-Bulli seam mines have been established by using desorption rate characteristic specific to each of those seams linked to the desorption rate of the benchmark Bulli seam gas content thresholds.

Over the ensuing twenty years:

- reased up to four fold
- coal production rates have incoutbursts have been acknowledged as a risk across all eastern seaboard underground coal mine districts
- there have been no fatalities or serious injury as a result of outburst since 1994
- the incidence of outburst have been reduced to less than a few per year. Those reported have occurred while remote mining or as a result of the failure to implement the OMP to design
- all outburst have occurred during development mining other than:
 - two reportedly low intensity outbursts on Longwall 23 at West Cliff Colliery on the 3rd of April 1998. The seam gas reported to be 98% CO2 and up to 21 m3/t (Harvey, 2002)
 - three outburst on the face of Longwall 27 at Metropolitan Mine on the 23rd of December 2016, 3rd and 4th of January 2017
- and importantly the industries tolerance for outburst and gas related hazards has been reduced significantly.

OUTBURST MECHANISM

Over the last three decades there have been numerous studies which have set out to improve our understanding of the outburst mechanism. Because of the wide variety of conditions under which outbursts occur, there is no single theory that can explain the phenomenon (Lama *et all*, 2002) The general accepted mechanism in Australia follows:

- outbursts in the Bulli, German Creek, Goonyella Middle and Bowen seams have generally always occurred on geological structure; or on mining induced cleavage in the Gemini Seam in the Leichhardt Colliery experience (Hanes)
- in the area surrounding the outburst prone structure, the permeability has been reduced to almost zero as a result of the high stress conditions around the structure. The high stress / low permeability conditions causing seam gas content in the area of the prone structure to remain high despite gas predrainage efforts
- also often associated with the structure are slickensides and the presence of mylonite fine crushed coal
 - which makes drilling and coring conditions difficult
 - allows for gas desorption rates to be enhanced significantly once reservoir pressure has been reduced to desorption pressure levels and
 - \circ takes no load, transferring stress to the surrounding coal
- as the mining face approaches the outburst zone:
 - the highly stressed coal between the face and the outburst structure takes on more stress and permeability is further reduced
 - the coal barrier between the mine face and the outburst zone is reduced to a critical thickness and fails as the effective stress exceeds the material strength of the coal
- upon failure of the coal:
 - the fluid pressure on the coal in and around the structure falls suddenly from above gas desorption pressure to atmospheric pressure
 - the seam gas desorbs rapidly, and
 - the free gas pressure in the coal increases rapidly
- the rate of pressure build up is dependent upon the rate of gas desorption and the volume of gas available:
- o gas desorption rates:
- increase with increasing gas content
- \circ have long been recognised to be greater for CO₂ than for CH₄
- o are enhanced by the presence of fine grained coal / mylonite.
- the volume of gas (and coal) available for ejection is defined by seam gas content and the extent of the structured zone
- the gas pressure generated by the rapidly desorbing seam gas promotes outward projection of the coal

OUTBURST THRESHOLDS

Since the early 90s the Australian underground coal industry has used seam gas content levels as outburst thresholds exclusively. Prior to that gas desorption rates were employed as an indicator of outburst risk (Lama, 1995) using the desorption rate defined by Hargraves Emission Value (EV) desorption meter which measured the volume of gas desorbed over a 2 - 6 minute period from a 4 g sample of coal taken from drilling cuttings off short boreholes (2-3 m) drilled immediately ahead of the development face. Thresholds defined by Hargraves were typically employed:

- 1.5cc/g for CH₄ and
- cc/g CO₂

Other than being unsuitable for high production mining the EV meter was unsuitable in a mixed gas environment and effected by moisture, variations in sample ash, and knowing where the sample was taken from.

Bulli Seam Thresholds

The gas content thresholds currently employed in Bulli seam mines were first defined by Lama (1995). Those thresholds developed with reference to:

- overseas outburst thresholds in mostly CH₄ rich coal seams o Poland, Russia, Germany, Bulgaria and China. In particular the 9.0 m³/t (CH₄) threshold employed at Germany's Ibbenburen Colliery
- thresholds employed at Collinsville Colliery of 5 m^3/t CO₂, desorbable gas content.

Lama's original thresholds were defined for desorbable gas content as measured by the slow desorption method of testing:

- 8 m³/t (CH₄) and 4 m³/t (CO₂) where structure was present
- 10 m^3/t (CH₄) and 7 m^3/t (CO₂) in the absence of structure.

These were subsequently modified with the introduction of the fast desorption method of gas content testing for total desorbable gas content:

- 9.4 m³/t (CH₄) and 6.4 m³/t (CO₂) where structure was present
- 12 m³/t (CH₄) and 10 m³/t (CO₂) in the absence of structure and accepting the occurrence of small outburst (< 20 – 40 tonnes).

In proposing the Bulli seam thresholds Lama (1995):

• makes reference to mathematical modelling which led him to conclude that the thresholds were appropriate for mine development rates up to 50 m /day

- suggested that for reduced development advance rates of between 10 and 12 m /day, the thresholds of 9.4 m³/t (CH₄) and 6.4 m³/t (CO₂) could be raised by a factor of 1.2, to 11.3 m³/t and 7.7 m³/t respectively
- indicated the thresholds include a factor of safety 19%
- presented a chart (Figure 1) gas content and composition from a dataset of measurements derived from Tahmoor and West Cliff Collieries over a three year period. The chart includes:
 - \circ $\;$ the proposed lines of outburst thresholds for structured and unstructured coal
 - gas content / composition measurements where development mining had taken place indicating:
 - no outburst had occurred where the gas content was measured to be less than 9.4 m³/t (CH₄) and 6.4 m³/t (CO₂)
 - a number of measurements were taken where the gas contents exceeded both threshold lines without the occurrence of outbursts
 - a number of outburst were recorded where the gas contents had exceeded the unstructured coal thresholds
 - ten outburst, varying in size (as defined by the amount of coal ejected) from zero up to between 30 to 40 tonnes where the gas content was measured to be between the two threshold lines. Lama indicating these outbursts were "too small to cause any major damage or endanger life of personnel"



Figure 1: Measured gas content close to outburst prone structures – Tahmoor and West Cliff mines (Lama, 1995)

Since their introduction mines operating in the Bulli seam have applied the thresholds with some differences:

- Appin Mine operates under a single threshold of 9.4 m^3/t (CH₄) and 6.0 m^3/t (CO₂).
- Tahmoor Colliery now employs three threshold lines based on the original work of Lama:

1) For unrestricted mining in structured coal the gas content has to be measured to be below 9.4 m^3/t (CH₄) and 6.4 m^3/t (CO₂)

2) Where the gas content is greater than the base thresholds but less than 11.3 m³/t (CH₄) and 7.7 m³/t (CO₂), normal mining is employed but with development advance rates limited to 12 m/day in structured coal

3) Where it is proven that the coal is unstructured, thresholds of 12 m3/t (CH4) and 10 m^3/t (CO2) are applied.

- Metropolitan Mine have recently modified their threshold limits to have two lines:
 - 1) 9.5 m³/t (CH4) and 6.4 m3/t (CO2) for unrestricted mining and
 - 2) 11.3 m³/t (CH4) and 7.7 m3/t (CO2) at development advance rates limited to 12 m/day.

Other seams

Gas content thresholds for non-Bulli seams have been set using the desorption rate of the specific seam being assessed relative to the desorption rate defined for the Bulli seam "bench mark" coals following the work undertaken by Williams and Weissman (1995) and Williams (1997). During gas content testing of Bulli seam samples from dominate CH_4 and CO_2 areas of West Cliff Colliery it was found that the desorption rate for coal having a measured gas content of 9.5 m³/t (100% CH₄) was the same as that where the gas content was measured to be 6.2 m³/t (100% CO₂). GeoGAS's fast desorption method of gas content determination has been subsequently used to set gas content thresholds using desorption rate index of 900 (DRI900). The DRI900 defined as the quantity of gas desorbed after 30 seconds of crushing a 150 g sample normalised to the total desorbable gas content of the full sample. The DRI900 has been used to define gas content thresholds for outburst mitigation for the:

- Wongawilli seam in the Illawarra Coal Measures to be circa 5.5 m³/t (98% CO₂)
- West Wallarah and Fassifern seams in the Newcastle coalfields to be about 10 $m^3\!/t$ (98% $CH_4)$
- Seams in the Hunter Valley coalfields to be in the order of 9 11 m³/t (CH₄ rich) and 6-7 m³/t (for CO₂ rich coals)
- Hoskisson seam in the Gunnedah Basin has an outburst threshold of about 6 m³/t (predominately CO₂)
- Goonyella Middle and Lower seams, the Harrow Creek Upper and Lower seams in the Moranbah Coal Measures are around 7 m^3/t at 98% CH₄
- Elphinstone and Hynds seams of the Rangal Coal Measures in the range of 7 to 8 m^3 /t (CH₄ rich)
- Newlands Upper seam of the Rangal Coal Measures to be about 9.5 m³/t at 98% CH₄
- Rangal coal measures seams toward the south of the Bowen Basin to be around 6 m³/t CH₄ rich and 4.5 m³/t at 60% CO₂.

Outburst control zones

The 2014 NSW Coal Mines Regulations introduced the concept of mining in Outburst Control Zones (OCZ). OCZ defined as any area of a mine where either:

- a. The gas content of the seam was measured to exceed 9 m^3/t (100% $CH_4)$ or 5 m^3/t (100% $CO_2)$
- b. Where the GeoGAS Desorption Rate Index (DRI) method is used—the desorption rate index of gas exceeds 900.

Where the area of the mine is defined as an OCZ mining in that area is deemed a High Risk Activity and the mine is required to submit a high risk activity application to the Department of Industry Resources and Energy prior to mining.

OUTBURST THRESHOLDS – MISCONCEPTIONS and CRITICISMS

Misconceptions

Permeability and raising thresholds for reduced mining rates

Though often referred to in technical papers and in OMPs, permeability like a number of other gas reservoir characteristics plays no direct role in outburst initiation. Assessing one mine as being more disposed to outburst due to a lower permeability regime (for the same gas content and structural fabric) is misleading. The lower permeability mine will take longer to predrain or alternatively cost more to predrain for the same drainage lead time. Once the gas content is reduced to target levels a seam with a permeability of 2 mD will be no more prone to outburst than one with a permeability of 200 mD. The catch is the change of permeability in and around prone structures; this is key to understanding outburst mechanics, the importance in defining structure and in developing compliance core testing strategies. It is also the reason why many consider the second tier thresholds proposed by Lama based on reduced development rates as flawed (Williams, 2011). The unsound logic of sneaking up on an outburst zone by way of reduced mining rates first proposed by Lama (1995) has been employed to lift outburst thresholds at Tahmoor and Metropolitan Mines. Those mines adopting Lama's 2nd tier thresholds of 7.7 m³/t (100% CO₂) and 11.3 m³/t (100% CH₄) in structured coal for development rates limited to 12 m/day (Wynn, 2011). That work has seen Black (2016) suggesting that miners operating in other coal seams might follow suit by employing a DRI of 1200. Whether seam permeability is typically in the range of 2-5 mD or 20-50 mD the permeability in and around an outburst zone on a prone structure such as a strike slip fault will approach 0 mD as a result of the stress associated with the structure. There are number cases through history where outbursts have occurred after crib breaks (Hargraves, 1975), during remote mining where advance rates are painfully slow, or on longwalls where the face has retreated less than 9 m over 12 days - a long way short of the 10 - 12 m / day proposed by Lama (1995).

There are numerous instances across the Australian underground coal industry where:

- borehole monitoring provides the first indication an area is not draining as per normal
- the first response is generally to infill the borehole pattern in an effort to promote gas drainage
- where there is prone structure present the permeability will be tight and gas production rates are typically as disappointing as those measured from the first array of boreholes
- cores are taken where drilling conditions permit and the gas content is often at virgin levels.

This is a scenario that is familiar to most that have mined in the gassy Bulli, German Creek or Goonyella Middle seams. The permeability will be approaching 0 mD in and around the outburst prone structure and hence the gas content remains high; where the zone can be drilled it can only be drained with boreholes at tight spacings (2-5 m) and long lead times (measured in months). Gas bleed off from these areas is so slow that any mining advance rate is too fast (Williams, 2011). Most mine operators will then navigate these areas using grunching or remote mining techniques.

Predicting outbursts

Through the 90's there was often debate regarding outburst size and then the terminology applied to define them. Terms used to describe outbursts included outbursts, bursts, slumps and bumps. The latter terms suggest low intensity events (albeit uncontrolled). There are a number of references in Lama's work (1995) suggesting outburst of less than 20 to 40 tonnes were harmless and possibly acceptable.

The industry has moved on and our tolerance to hazards associated with seam gas, particularly those involving an uncontrolled release of gas, has been reduced significantly since the 90's. Other than the obvious, the underlying concern with accepting any form of uncontrolled event is that we can predict the size of an outburst. Structures are regularly mined through at elevated seam gas contents without outburst, yet there is no way of saying that the next time the same structure is mined through an outburst will not occur; the most recent example being the Metropolitan longwall outbursts. The outburst could be 40 tonnes or hundreds of tonnes, the volume of gas released could be a couple of hundred cubic metres or tens of thousands of cubic metres. Our capability to identify and map

structure is improving but our ability to predict which structure will outburst and what the size the outburst might be with any confidence is not where we might like it to be.

Criticisms

Criticisms of the current outburst gas content thresholds are few but typically they are critical of their simplicity or that they are overly conservative:

"The concept of a single measurement being an indicator of whether a coal seam is outburst prone might be convenient but is not valid" (Gray and Wood, 2013).

"The attempt by the Australian mining industry to shoehorn all of our outburst risk assessment on to a single gas content measurement is a gross simplification" (Gray and Wood, 2013).

"The need for a better approach is brought about by the simplistic and indeed incorrect nature of what is being used in Australia at present. This generally, but not invariably, leads to overly conservative gas drainage practice" (Gray and Wood, 2013).

These criticisms seem to disregard that the gas content thresholds employed across the industry are just one element of a risk management system which has unequivocally proven to be an effective mitigation strategy since implemented in the mid 1990's.

Other aspects often referred to by the critics as not being included in the threshold are in fact taken into account when either assessing a mine or an area within a mine of the risk of outburst and other seam gas hazards:

- during the initial mine feasibility studies coal strength, stress, seam lithology, presence and type of structure, gas content and quality, gas desorption rate and permeability
- panel hazard management plans, longwall gas management plans and each of the outburst Authority to Mine notices consider – geological structure, stress, seam lithology, previous mining history and mine plan, drilling history, gas content and quality, gas drainage performance, gas emission history and forecasts.

Calls of conservatism need to be considered in line with the fact that most mines predrainage programs will target remaining gas content levels well below outburst threshold limits in either the Sydney or Bowen Basin mines to minimise gas exceedances on either gate road development (between 3 and 5 m³/t) or longwall extraction (< 3 m³/t).

The use of gas content thresholds in conjunction with the other essential elements of the management system has proven to be effective. Gas content measurements are practical within the mining process and provide a well understood indication of energy available (albeit a static measurement). It's relatively simple:

- remove the energy through gas predrainage
- measure the gas content to confirm the predrainage plan
- consider the seam geology, drilling and gas drainage performance, mining plan and history
- authorise mining where it is safe to do so.

It could be argued that outburst gas content thresholds employed in the Hunter Valley and Bowen Basin mines are unproven given their origins. Outbursts that have occurred in the Bowen Basin since the implementation of OMPs at Central Colliery (July 2001) and North Goonyella Mine (October 2001 and May 2012) have all been relatively low energy events and have each occurred as a result of the failure to apply the OMP as designed. Gas content measurements at each of these sites post outburst were at or above outburst threshold levels; and hence there is no clear evidence to raise or lower gas content thresholds as an outcome of investigations into these few outbursts.

Without the occurrence of a statistically reliable number of outbursts it is always going to be difficult to modify outburst thresholds, regardless of how good the science might be. While we continue to target higher longwall production rates and faster gate road development rates the only foreseeable movement in gas predrainage targets appears to be downward. The likelihood of a future carbon tax can only place more pressure on lower gas drainage targets.

Criticism of the use of gas desorption rate as a means of setting gas content thresholds in non-Bulli seams are similarly poorly founded:

"Related to errors in gas content measurement" (Gray and Wood, 2013) "Based on pseudoscience fitting a straight line to some group of data without having thought through the measurement process and the errors it contains" (Gray and Wood, 2013) "The outburst threshold limit for this dataset is the gas content value at the point where the DRI of 900 meets the average minus 2 x SD line" (Black, 2016)

The limits of accuracy of gas content measurement are now well understood and accepted in their application in gas reservoir modelling and seam gas management. Gas desorption rate has long been acknowledged as being fundamental in the outburst initiation process. The measurement of gas desorption rate from a sample of coal during the fast desorption method of gas content testing is effected by the inaccuracies inherent in the gas content testing. Desorption rate is also effected by sample moisture and by the consistency of the crushing process; hence we see a scatter around the (mean) line fitted to the gas content / desorption rate data set for a particular seam and gas composition.

The mean line fitted through the data set is used to define the outburst threshold. The mean less two standard deviations was originally intended by Williams to define the gas content limit for which gas predrainage would be initiated for outburst mitigation. For example the outburst threshold for the Goonyella Middle (GM) seam is typically set at about 7 m³/t (100% CH₄). The gas content for the GM seam defined by the mean less two standards deviation of circa 6.2 m³/t is for practical purposes is meaningless as all mines operating in the GM seam target remaining gas content levels of 3 m³/t and most mines in the same thick seam typically commence predrainage when the gas content is in the range of 4 to 5 m³/t.

Acknowledging the scatter in the correlation between gas content and desorption rate due to the effects of sample moisture and potential variability in crushing, Williams's (1995, 1997) intended use of the DRI900 was only ever as a means of setting gas content thresholds based on the benchmark Bulli seam desorption characteristics. Once set, all outburst assessments for that particular seam should be based on the defined gas content thresholds with the DRI value that accompanies gas content measurements from GeoGAS used as supplementary data only.

The use of the DRI as prescribed in the 2014 NSW Coal Mines Regulations for defining Outburst Control Zones suffers from the same potential limitations as described above when using it as a standalone indicator of outburst proneness – sample moisture and laboratory processing limitations. For the Bulli seam the OCZ defined by gas content - method (a) in the regulations would be slightly different to that defined by the DRI 900 (method (b)). For non-Bulli seams the use of method (a) is not valid, and using method (b) DRI900 would give and OCZ the same threshold as the outburst threshold which is inconsistent with the difference in OCZ limits and outburst thresholds limits in the Bulli seam.

OUTBURST THRESHOLDS and OUTBURT MANAGEMENT – IN CONTEXT

It is indisputable that the risk management systems employed by the industry to manage the risk of outburst since the mid 1990's has been effective. During a period where mine production rates have

increased by millions of tonnes per annum, the number of outburst that occur has been reduced significantly. There have been no fatalities or serious injuries as a result of outburst in Australia since their implementation more than 20 years ago. Other than the outburst that have occurred during grunching or remote mining, where the elevated risk has been identified prior to mining, those outbursts that have occurred during normal development mining have been as a result of failure to implement the plan. The recent outbursts on the longwall face at Metropolitan are the only exception to this and provide an opportunity to revisit our outburst management strategy for that phase of mining.

Criticisms of the use of the gas content threshold seem to disregard the fact that it is just one element within a management system, and that the other factors that can been used to define the risk of outburst such a structure, stress, coal strength etc, are in fact taken into account throughout the different phases of mine planning and authorisation.

Given what we have learnt about seam gas and gas drainage over the last 30 years it evident that the original Bulli thresholds proposed by Lama were not scientifically based but borrowed from experience elsewhere and modified slightly based on Bulli seam experience. That said the thresholds are part of a risk assessment system that works. Arguments regarding conservatism of thresholds neglect to consider the gas drainage targets required to meet statutory limits on gate road development or to produce at longwall production rates of 2 - 10 Mtpa.

Accepting our current understanding of the outburst mechanism, and taking the lessons learnt from our gas drainage and mining experience, it is clear that there is no feasible mining rate slow enough to allow gas to bleed off from an undrained outburst prone structure in a time frame that would render the outburst structure benign. Employing a higher gas content threshold justified by a development rate reduced to 12 m / day is flawed.

Though the use of gas desorption rate to transfer proven gas content thresholds to non-Bulli seams may not satisfy the scientific requirements of a few, gas desorption rate has long been acknowledged as key in outburst initiation. For a given gas content and composition for coal samples from the same seam, reported desorption rates will vary slightly due to inaccuracies inherent in the gas content testing, sample moisture, variations caused by sample selection and by inconsistency of the crushing process. These shortcomings in determination of the DRI are understood; how it is applied is important. The use of the DRI should be limited to setting gas content thresholds and providing supplementary evidence only in assessment of outburst risk. A single measurement of DRI should not be used as a standalone indicator of outburst risk.

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CONTROL AND MANAGEMENT OF GAS EMISSIONS TO REDUCE PRODUCTION DELAYS AND FUGITIVE EMISSIONS

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ABSTRACT: The process of longwall coal extraction coal causes fractures in the overlying and underlying strata and these fractures become pathways for gas released from adjacent coal seams to flow into the mine workings and contaminate the ventilation air. If the rate of gas emission exceeds the diluting capacity of the ventilation air, the gas concentration will increase and exceed the statutory limit resulting in production delays.

All potential gas sources within the planned mining area, including coal seams located above and below the working seam, should be identified and sufficient gas data collected and used to determine the specific gas emission from each gas source. Gas reservoir and emission modelling is recommended to determine specific gas emission and changes in gas emission from individual sources over the planned mining area. Accurate gas reservoir and emission modelling provides the information required to accurately design gas drainage programs to effectively manage gas emissions and minimise the risk of 'gas-outs'.

INTRODUCTION

To effectively control and manage gas emissions in an underground coal mine, sufficient information must be collected to enable all potential gas emission sources to be identified. Each potential source should be investigated to determine the volume and rate of gas release during mining operations. There are many potential sources of gas emission in an underground coal mine (Black and Aziz, 2009), as indicated in Figure 1, which includes:

- A. Emission from exposed mine roadways;
- B. Emission during coal cutting both development and longwall;
- C. Emission into longwall goaf from adjacent gas bearing coal seams and strata;
- D. Emission from longwall goaf into connecting airways; and
- E. Emission from coal being transported from mine via the coal clearance system.



Figure 1: Sources of gas emission in a typical underground longwall coal mine

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Mine operators are required to maintain a safe workplace and provide safety management systems which include provisions to maintain gas concentrations within all accessible places within the mine below prescribed statutory limits (WHS Regulation, 2014). In cases where the mine has not identified and assessed all potential gas sources and insufficient capacity has been provided in the ventilation and gas management systems, the risk of exceeding the statutory gas concentration limits is greatly increased.

Incorporating gas reservoir analysis and gas emission forecasting into the mine planning process provides a means of forecasting the return airway gas concentration for periods defined in the mine production schedule. This process allows the impact on gas concentration, from varying the level of pre-drainage and goaf drainage, to be assessed.

The steps from initial exploration data collection through to forecasting the return airway gas concentration, including pre-drainage to reduce the specific gas emission and goaf drainage to reduce the volume of gas released into the longwall ventilation system, are listed below:

- Exploration (Data Collection)
- Gas Reservoir Modelling
- Specific Gas Emission Calculations
 - Specific gas emission is effected by mine design i.e. longwall face width and cut height
 Pre-drainage may be used to reduce specific gas emission
- Longwall Gas Make Calculations
 - Longwall gas make is effected by production rate i.e. high production equals high gas make
- Return Airway Gas Concentration
 - Gas emissions and return airway gas concentration is effected by goaf gas extraction rate i.e. removing gas from the goaf prior to being released into the mine ventilation system;
 - Ventilation air quantity dilutes gas emissions therefore has some effect on reducing return airway gas concentration.

EXPLORATION DRILLING AND DATA ACQUISITION

Data collection during exploration programmes has a significant impact on determining the size and significance of the gas reservoir, and identifying specific areas and coal seams that may require gas drainage as a pre-emptive action to reduce gas emissions when the area is mined.

Quite often exploration tends to focus on the working seam and little information is collected from coal seams and other gas bearing strata that may be present above and below the working seam. Although exploration drilling will pass through coal seams above the working seam, core samples are not regularly collected for gas testing and proximate analysis, and logging may not accurately pick the level to the top and base of each coal seam. It is also quite common that the total depth of exploration boreholes will not extend further than approximately ten metres below the base of the working seam. In areas where coal seams are present below the working seam, the absence of exploration data prevents accurate assessment of the impact of gas emissions during longwall extraction. In areas where coal seams may be present below the working seam, it is recommended that exploration boreholes extend approximately 40-50 metres below the target working seam and the depth and thickness of all coal seams in the sequence are identified and recorded.

Figure 2 illustrates the difference between two exploration boreholes; (A) the total depth of the borehole extends approximately 80 metres below the base of the working seam (D seam) and all coal seams have been accurately logged, and (B) the total depth of the borehole extends less than ten metres below the base of the working seam, no information has been collected to allow an assessment of gas emission potential from underlying coal seams and limited detail is available from overlying coal seams.



Table 1 lists gas reservoir data collected from exploration borehole BH223, which includes data collected from five coal seams above, and five seams below, the D seam.

Figure 2: Use of exploration drilling to accurately identify and collect data from coal seams with gas emission potential

Table 1: Gas reservoir information	collected from	exploration borehole BH223
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GAS BEARING STRATA		Depth to Seam Roof (m)	Depth to Seam Floor (m)	Seam Thickness (m)	RD (t/m ³)	Gas Comp. %CH ₄	Distance Above/Below Working Seam (m)	Virgin Gas Content (m ³ /t)
	A Upper Seam	335.90	336.62	0.72	1.44	100	78.5	4.37
DOOL	A Lower Seam	342.69	344.23	1.54	1.30	100	70.9	4.50
SEAMS	B Seam	377.16	381.20	4.04	1.38	100	33.9	5. 1 5
02.000	C Seam	389.49	390.38	0.89	1.30	100	24.7	5.87
	D Upper Seam	398.12	398.90	0.78	1.32	100	16.2	6.02
	D Seam	415.08	417.20	2.12	1.31	100		7.30
	E Upper Seam	432.73	433.73	1.00	1.38	100	15.5	7.50
51.000	E Seam	433.73	435.12	1.39	1.37	100	16.5	7.61
FLOOR SEAMS	F Seam	454.17	457.34	3.17	1.32	100	37.0	7.48
	G Seam	476.41	478.89	2.48	1.27	100	59.2	7.89
	G Lower Seam	488.21	489.54	1.33	1.35	100	71.0	8.03

The number of exploration boreholes planned to be drilled in an exploration programme should cover the planned mining area and the spacing between boreholes should be close enough to identify changes in reservoir characteristics. In cases where unexpected and unusual results are obtained from an exploration borehole, additional drilling should be planned near that borehole to check the accuracy of the previous results and collect additional data to assist in detailing the changes to the gas reservoir in those areas.

Figure 3 shows the location of twenty-two (22) exploration boreholes drilled over a planned longwall mining area. In this example, no exploration boreholes have been drilled over the final four (4) longwall panels. The layout of most longwall mines is such that mining commences in relatively shallow conditions and, during the mine life, mining depth and the gas content of the coal seam tends to increase. It is important that exploration drilling and data collection is completed well ahead of planned mining and provides sufficient time to (a) assess the gas reservoir, and (b) design and

implement effective gas management and emission reduction measures to control forecast gas emissions.



Figure 3: Location of exploration boreholes covering the planned longwall mining area

GAS RESERVOIR ANALYSIS

Analysis of the gas reservoir is recommended to identify the relative impact of all potential sources of gas emission within the goaf and to evaluate the impact of pre-drainage to remove gas from specific coal seams to reduce total Specific Gas Emission (SGE), prior to longwall extraction.

Accurate analysis of the gas reservoir requires a comprehensive record of results from gas testing and proximate analysis of coal samples collected from all coal seams intersected during drilling of a sufficiently large number of exploration boreholes covering the planned mining area. In cases where reservoir data is not available for specific coal seams and/or the spacing between exploration boreholes is too great, the accuracy of the gas reservoir analysis decreases due to reliance on interpolation between known data points.

The total SGE (m^3/t) is calculated at each exploration reference borehole location and requires the addition of SGE values calculated for each coal seam located above and below the working section that will release gas into the goaf following longwall extraction. In addition to the reservoir information listed in Table 1 and details of the width and height of the mining excavation, the caving angles above and below the working seam are used to calculate the degree of gas emission (%) from each of the overlying and underlying coal seams. In this example, the Flugge Goaf Caving Model (MEA, 2006), is used to calculate the degree of emission. The equation presented below, Equation 1, incorporates the Flugge degree of emission calculation (MEA, 2006), and is presented as the method to calculate the SGE contribution (m^3/t) from individual coal seams. The total SGE at each reference borehole location is calculated by adding the SGE contributions from each coal seam in the sequence.

$$SGE_{i} = \frac{\left((Q_{M} - Q_{D} - Q_{R}) \times H_{i} \times RD_{i}\right) \times \left(\left((D_{LW} - (2 \times d_{i} \div \tan \beta)) \times 100\right) \div D_{LW}\right)}{(H_{LW} \times RD_{LW})}$$

Where:

 Q_M is the measured virgin coal seam gas content; Q_D is the gas content reduction by gas drainage;

 Q_R is the residual gas content of the coal seam, post longwall extraction;

 H_i is the thickness of the coal seam;

 RD_i is the relative density of the coal seam;

 D_{LW} is the width of the longwall face;

 d_i is the distance of the coal seam above/below the working seam section;

 β is the caving angle of the longwall goaf above/below the working seam section;

 H_{LW} is the height of the working seam section; and

 RD_{LW} is the relative density of the working seam.

Figure 4 shows the gas content and SGE contribution of each coal seam intersected by borehole BH223. In this example, there has been no pre-drainage to reduce gas content below virgin levels, and the total SGE is $20.5 \text{ m}^3/t$. The greatest individual SGE contributions are from D seam (6.5 m $^3/t$) and B seam (5.2 m $^3/t$). Figure 4 also shows that if no data was collected below the D seam, the SGE would be incorrectly calculated at 15.1 m $^3/t$.



Figure 4: SGE contribution by coal seam with no pre-drainage to reduce gas emissions from individual coal seams

Figure 5 shows contours of the total SGE expected when mining each longwall panel and these values are based on no pre-drainage to reduce gas content prior to mining. Underground coal mines will typically commence pre-draining the working seam in areas where the gas content is above 5.0 to 6.0 m^3 /t. There may be a number of factors that necessitate the use of pre-drainage which may include (a) reducing rib emissions to maintain the gas concentration of intake ventilation air below 0.25% CH₄, (b) reducing the gas content of the working seam below defined outburst threshold gas content limits, and (c) reducing the gas content of the working seam to avoid high gas emissions during the coal cutting cycle that may deenergise the cutter head and face equipment due to gas concentrations exceeding prescribed maximum concentrations. The results presented in Figure 4 show relatively high SGE from both B and D seams therefore the impact of pre-drainage to reduce the gas content of these two seams should be considered.

Figure 6 shows the impact of pre-draining the B seam to 4.0 m^3 /t and the D seam to 3.0 m^3 /t which achieves a 30% reduction in SGE at borehole BH223, reducing the SGE from 20.5 m³/t to 14.2 m³/t. The reduction on total SGE, achieved through pre-draining the D seam and parts of the B seam, is shown in Figure 7.



Figure 5: Specific Gas Emission (m³/t) with no pre-drainage to reduce total gas-in-place and emissions from individual coal seams



Figure 6: SGE contribution by coal seam after pre-draining B seam to 4.0 m³/t and D seam to 3.0 m^3 /t



Figure 7: Specific Gas Emission (m³/t) after pre-draining B seam to 4.0 m³/t and D seam to 3.0 m^3/t

Details of the gas reservoir data and emissions calculated for each coal seam intersected by borehole BH223 are listed in Table 2.

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GAS B	EARING STRATA	Depth to Seam Roof (m)	Depth to Seam Floor (m)	Seam Thickness (m)	RD (t/m ³)	Gas Comp. %CH ₄	Distance Above/Below Working Seam (m)	Virgin Gas Content (m ³ /t)	Gas Drained Pre-Mining (m ³ /t)	Gas Content (m ³ /t)	Potential Gas Emission (m ³ /t)	Mass of Coal in Seam (t/m ²)	Gas Emission (m ³ /m ²)	Degree of Gas Emission (%)	Specific Gas Emission (m ³ /t)
	A Upper Seam	335.90	336.62	0.72	1.44	100	78.5	4.37		4.30	0.07	1.04	0.07	62.29	0.02
0005	A Lower Seam	342.69	344.23	1.54	1.30	100	70.9	4.50		4.30	0.21	2.00	0.41	65.94	0.10
SEAMS	B Seam	377.16	381.20	4.04	1.38	100	33.9	5.15	1.2	2.02	1.98	5.56	10.99	83.71	3.31
020	C Seam	389.49	390.38	0.89	1.30	100	24.7	5.87		1.71	4.16	1.15	4.81	88.13	1.52
	D Upper Seam	398.12	398.90	0.78	1.32	100	16.2	6.02		0.85	5.17	1.03	5.31	92.22	1.76
	D Seam	415.08	417.20	2.12	1.31	100		7.30	4.3	0.78	2.22	2.78	6.18	100.00	2.22
	E Upper Seam	432.73	433.73	1.00	1.38	100	15.5	7.50		0.80	6.69	1.38	9.21	70.73	2.34
FLOOD	E Seam	433.73	435.12	1.39	1.37	100	16.5	7.61		1.31	6.30	1.91	12.01	68.85	2.97
SEAMS	F Seam	454.17	457.34	3.17	1.32	100	37.0	7.48		7.48	0.00	4.19	0.00	30.32	0.00
0EAWO	G Seam	476.41	478.89	2.48	1.27	100	59.2	7.89		7.89	0.00	3.15	0.00	0.00	0.00
	G Lower Seam	488.21	489.54	1.33	1.35	100	71.0	8.03		8.03	0.00	1.79	0.00	0.00	0.00
TO	TOTAL SPECIFIC m ³ gas per tonne of coal produced						14.24								
G	AS EMISSION	litres gas p	per tonnes of	coal produce	ed										14,240

Table 2: Gas reservoir information and gas emission calculation results for exploration
borehole BH223

PRE-DRAINAGE

Figure 8 shows the level of gas content reduction required from pre-drainage to reduce the gas content of the D seam from virgin levels to the 3.0 m^3 /t pre-mining gas content target. In this example, the B seam has also been identified as a target for pre-drainage with a target to reduce the gas content to below 4.0 m^3 /t. The virgin gas content of the B seam is presented in Figure 9 and the contours show that, up to the limit of current exploration data, there is a relatively small area where gas content exceeds 4.0 m^3 /t therefore only a small area requires pre-drainage.



Figure 8: Gas content (m^3/t) to be removed from D seam by pre-drainage to achieve the target 3.0 m³/t pre-mining gas content



Figure 9: Gas content of the B seam (m^3/t) – contours indicate areas greater than 4.0 m³/t that require pre-drainage

Drilling boreholes into coal seams to pre-drain gas prior to mining is achieved by (a) surface drilling methods, commonly referred to as Surface-to-Inseam (SIS) drilling, (b) underground drilling methods, commonly referred to as Underground-to-Inseam (UIS) drilling, or (c) a combination of both SIS and UIS drilling methods (Black and Aziz, 2009). Compared to UIS drilling, SIS is high cost and to achieve a reasonable return on investment, it is necessary to drill SIS boreholes many years ahead of planned mining to provide a minimum 3 to 5 year drainage time. In comparison, UIS boreholes typically have a short life and provide drainage for 6 to 12 months. Therefore, UIS gas drainage requires a greater number of closely spaced boreholes to achieve effective gas content reduction over the short drainage period.

Often, a combination of SIS and UIS drilling will be used, SIS initially to drain gas from areas that are beyond the reach of UIS drilling, followed by UIS to drain areas where the gas content remains greater than the target pre-mining gas content threshold levels. In addition to drainage time and drilling cost, there are many factors, such as those listed in Table 3 that should be considered during

the process of selecting a drilling method(s) to effectively and efficiently pre-drain gas from a coal seam (Black and Aziz, 2010). The gas drainage characteristics specific to the coal seam to be drained should be understood and the gas drainage program designed to achieve maximum gas drainage efficiency (Black and Aziz, 2011).

Gas content	Gas content reduction required by pre-drainage
Gas composition	Mining schedule and available drainage time
Degree of saturation	Area to be drained and distance from existing mine workings
Permeability	Potential changes and uncertainty of the mine plan
Seam thickness	Depth to target coal seam
Seam dip	Geological structures
Cleat and stress orientation	Surface access for drilling and gas management infrastructure
Coal type and rank	Underground access for drilling and gas management infrastructure

Table 3: Factors to be considered in the selection of pre-drainage drilling methodand drilling patterns

GAS EMISSION ANALYSIS

The gas emission rate is a measure of the volume of gas that will be released from the combined gas sources during coal production operations at the mine. The average gas emission rate in this example is calculated based on an average production rate of 1,000 tonnes per hour. Average annual production is 4.0 Mtpa, there are 50 planned production weeks per year, and 80 production hours per week. The average longwall gas make resulting from an average production rate of 1,000 tonnes per operating hour, having pre-drained the B and D seam to 4.0 m³/t and 3.0 m³/t respectively, as discussed above, is presented in Figure 10. In addition to pre-drainage to reduce SGE, additional actions that may be taken by the mine operator to reduce the gas concentration in the ventilation air include (a) increase the ventilation air quantity, (b) reduce the production rate, and (c) drain gas from the goaf to reduce the gas volume that would otherwise report to the ventilation system. Reducing production rate is generally not desirable and there is typically limited capacity available within the mine ventilation system at most mines to support increasing the ventilation quantity supplied to the longwall panel. Therefore, most effective option available to mine operators to reduce gas emissions, in addition to more intensive use of pre-drainage, is to utilise efficient goaf gas drainage systems.



Figure 10: Average Longwall Gas Make (L/s) after pre-draining B seam to 4.0 m³/t and D seam to 3.0 m³/t

When designing systems to manage longwall gas emissions, the required capacity of the system should be based on expected 'peak' gas emissions. As the name suggests, the 'peak' gas emission rate is greater than the 'average' gas emission rate and tends to occur for relatively short periods before returning to the average rate characteristics for that specific mining area. The ratio of peak to average emissions will be confirmed through operating experience. However, in lieu of operating experience, designing the ventilation and gas management systems based on an emission rate of 1.5 x Average is considered reasonable.

In this example, 60 m³/s of ventilation air is directed across the longwall face and exits the longwall panel via the tailgate roadway. If the target maximum gas concentration in the longwall return air is 1.0% CH_4 then the ventilation air has the capacity to dilute a maximum gas emission rate of 600 litres per second. Therefore, to maintain the gas concentration below the nominated maximum value of 1.0% CH_4 , the mine will be required to provide and maintain goaf gas extraction systems that are designed to efficiently remove gas from the goaf such that the rate of gas emission into the ventilation system does not exceed 600 litres per second.

Figure 11 shows contours of the goaf gas extraction rate (litres per second) required to control goaf gas emissions to support longwall production at a rate of 1,000 tonnes per hour whilst maintaining the return airway gas concentration at 1.0% CH_4 , based on a peak-to-average gas emission ratio of 1.5 and having pre-drained the B seam to 4.0 m³/t and the D seam to 3.0 m³/t prior to longwall extraction.



Figure 11: Goaf drainage rate (L/s) to maintain TG gas to 1.0% CH₄ (Peak) – B seam drained to 4.0 m^3 /t and D seam drained to 3.0 m^3 /t.

Increased resolution can be achieved in the gas emission forecast through coupling the gas reservoir model and the mine production schedule. Provided there are sufficient exploration boreholes to provide confidence in the resolution and accuracy of the gas reservoir model, coupling the gas reservoir model with the production schedule will provide a forecast of gas emissions corresponding to the variable production rates presented in mine production schedules. The effect on gas emissions and gas concentrations resulting from varying the pre-drainage and goaf drainage intensity in various parts of the mine that align with specific periods in the production schedule may also be investigated.

The coupled model produces a forecast of future gas emissions that may be used to identify periods of high gas emission where general body methane concentration is expected to exceed statutory limits. In addition to modelling and assessing the impact of varying pre-drainage on reducing SGE, the coupled model is also used to assess the impact of varying the rate of gas extraction from the goaf

(goaf drainage) to further reduce total longwall gas emission into the ventilation air and the resulting impact on general body methane gas concentration in the ventilation air.

Goaf Drainage

Similar to pre-drainage, the extraction of gas from the goaf, commonly referred to as goaf drainage, may be achieved through drilling boreholes from both surface and underground drilling sites. Given the larger capacity of surface drilling rigs, boreholes drilled from the surface tend to be larger diameter and support greater gas extraction rates. Some Australian underground coal mines operate goaf drainage systems that consistently extract greater than 9,000 litres per second of CH_4 from the goaf to support longwall production rates of 8.0 to 9.0 Mtpa.

Drilling large boreholes, typically minimum diameter of 10 inch, in advance of the retreating longwall panel, at a spacing that may vary between 50 metres to 200 metres along the length of the longwall panel and typically offset 30 to 40 metres from the tailgate pillar, is the most common method used for goaf drainage in Australian underground coal mines (Black and Aziz, 2009). An alternative surface-based drilling method trialled at several Australian mines, utilises medium-radius drilling (MRD) directional drilling technology to drill one or more long laterals into the strata above the longwall block (Black and Aziz, 2009).

In areas where surface access is restricted, UIS drilling may be used to support or replace surfacebased drilling and gas extraction. The smaller diameter of UIS boreholes does limit gas extraction through each borehole therefore, to increase total goaf gas extraction, an increased number of boreholes is required, and if conditions allow, the boreholes may be reamed to a larger diameter.

Gas extraction through goaf seals is also an option however due to the tendency for air to be drawn around the perimeter of the active longwall goaf, goaf drainage through longwall seals typically draws low purity gas.

GAS MANAGEMENT AND FUGITIVE EMISSION REDUCTION

Effective gas management and reduction of fugitive emissions from coal mines relies on quality technical investigation to identify and assess the significance of all potential sources of gas emission associated with the mine operations and to design effective systems to control and reduce emissions. Management commitment is crucial to achieving effective gas management and the reduction of fugitive emissions. Corporate policies and standards should clearly state the goals and objectives, and the commitment to achieving the stated goals and objectives should be reinforced through setting measurable performance targets for executive and management personnel at all levels within the organisation. Companies must also demonstrate their commitment by providing funds necessary to achieve the stated performance targets. Coal mine gas management and ventilation systems should be designed and managed to ensure the gas concentration limits specified in coal mine health and safety legislations are not exceeded and visible commitment to reducing fugitive emissions should be demonstrated.

In addition to the use of gas drainage to reduce gas emissions contaminating the mine ventilation system, maintaining high standards throughout the mine ventilation network will have a significant impact on reducing gas emissions into the mine ventilation air. For example, regular monitoring and inspections of mine seals installed to isolate mined areas from current workings, should include gas testing and visual inspection to identify any evidence of gas leakage and if identified, prompt action should be taken to stop any identified leaks.

Installing equipment on the surface of an underground coal mine to flare drained gas has a significant impact on reducing fugitive emissions. Drained gas may also be used as a fuel source for reciprocating engine driven power generation units.

CONCLUSION

To reduce the risk of high gas emissions that adversely affect mine production, thorough analysis of the gas reservoir, including detailed exploration and gas reservoir data collection, is required to quantify the volume of gas contained within the reservoir and the specific gas emission expected in each mining area. Incomplete exploration significantly increases the risk of underestimating the size of the gas reservoir that may lead to the gas drainage and fugitive emissions management systems being inadequate to support planned mine production.

In addition to providing a measure of gas emission from all potential sources impacted by coal extraction, gas reservoir modelling can be used to assess the impact of pre-drainage on specific gas emission. Coupling the gas reservoir model to the mine production schedule will produce a gas emission forecast that may be used to calculate the expected average and peak gas concentration in the ventilation air. If the gas emission forecast indicates periods where the gas concentration exceeds statutory limits, the impact of goaf drainage and additional pre-drainage to reduce both gas concentration in the ventilation air and fugitive gas emissions from the mine can be assessed.

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A STUDY OF POTENTIAL OCCURRENCE OF BIOGENIC METHANE IN COAL SEAMS

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ABSTRACT: A significant proportion of the total gas emitted from coal mining, particularly for shallow seams (<300 m depth), is believed to have been generated from microbial activities within the coal seams and water filling the pores and fractures in coal. To investigate the potential and extent of gas generation in coal due to microbial activities, we developed a method to culture and monitor the production of biogenic methane in coal. We then applied the method to study the process of biogenic methane generation in coals from a mining region in New South Wales. Fresh coal core samples were collected from an exploration borehole drilled into a sequence of coal seams at a greenfield site where five coal seams were located between the depths of 50 to 250m. The formation water was collected from an adjacent borehole drilled into the same sequence of coals. The coal samples were crushed and mixed with formation water and other solutions in glass vials, and then placed in predesigned incubator at in-situ temperature to allow the production of methane over the life of the project. The results of measurements show that biogenic activities take place and that methane is generated. Methane continued to be produced throughout the life of the project for the studied coals.

INTRODUCTION

Primary origin of gas in coal is the result of coalification, which occurs under high pressure and temperature conditions generating thermogenic gas. This gas consists predominantly of methane (CH_4) , carbon dioxide (CO_2) , potentially some higher hydrocarbons such as ethane (C_2H_5) , and nitrogen (N_2) . However, a major source of gas, particularly at shallow depths, is believed to be through microbial activities and is labelled biogenic gas.

The occurrence of gas (mostly CH₄) of potentially biogenic origin has commonly been reported in Australian coalfields (see for example Smith *et al*, 1982; Smith and Pallasser, 1996; Boreham *et al.*, 1998; Faiz *et al*, 1999; Faiz and Hendry, 2006; Faiz *et al*, 2007; Li and al., 2008; Flores *et al.*, 2008; Formolo *et al.*, 2008; Hamilton *et al*, 2014). Numerous overseas researchers have also reported on biogenic processes for the generation of coal seam gas (see for example Rice, 1993; Whiticar, 1994, 1996; Clayton, 1998; Rice *et al*, 2008; Green *et al.*, 2008; Moore, 2012; Ritter *et al*, 2015; Park and Liang, 2016).

Overall it is believed that CH_4 is generated as a result of the microbial degradation of coal. Macromolecules in coal are broken down during acetate fermentation, providing nutrients essential for microbial metabolic functions. The extent of gas produced by microbial activities depends on the type of methanogenic microorganisms present in the coal seams and whether the in-situ conditions are aerobic or anaerobic, which in turn depends on the coal seam depth. Other factors believed to influence the biogenic gas production are redox potential and pH levels, temperature and coal properties such as porosity, composition and rank.

A new laboratory method for monitoring the generation of CH_4 in coal through microbial activities is developed to simulate the *in situ* process of biogenic generation of gas in coal seams. Using this method the production of biogenic CH_4 is accelerated so that the process of methane generation can be assessed in reasonable time periods (months). Details of the new method is described in the

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following sections, followed by the descriptions of coal core samples used, generated data analyses and interpretation of the results.

METHODOLOGY

Field procedure

The field procedure consisted of collecting fresh coal cores and formation water from a sequence of coal seams traversed by the designated surface drilling for this study. A suitable greenfield site was selected for the project where a sequence of coal seams could be intercepted by the surface exploration boreholes so that the effect of coal properties variation could also be investigated. Core coal samples were placed in CSIRO-designed, purposely built stainless steel gas tight canisters to allow rapid and secure sealing of the sample and inertisation of the headspace by flushing helium gas (He) into and out of the canister. The formation water was also collected from the same borehole, and from other boreholes in the vicinity of the sampled holes, to be used with coal samples in a predesigned microbial culture procedure. Formation water was placed in glass laboratory fluid containers, which were modified to suit the requirements of water collection at the drilling site. Formation water provides nutrients and supports metabolic activities of microbial populations living in coal reservoirs, leading to the generation of various gases. Specific procedures were designed for collecting and testing the required coal and water samples. Both coal and water sampling containers were fully sterilised before use in the field to eliminate the risk of introducing microbial populations other than those from the reservoir. For the anaerobic microbial populations to survive, oxygen (O_2) was removed from the water by adding deoxygenating solutions to the collected formation water as well as by bubbling a stream of ultra-pure He through the water for a sufficient amount of time.

Laboratory procedure

Once the coal samples were brought to the laboratory, they were crushed and pulverised in a helium atmosphere using a purposely built crusher, on which the stainless steel canister containing coal could be mounted and coal crushed without opening the canister. For some samples, the molecular and isotopic composition of gas desorbed during crushing was analysed. Crushed coals were then removed from the canisters, inside a sterilised, anaerobic chamber filled with helium gas, and partitioned into multiple small subsamples. The coal subsamples were mixed with other solutions and placed in gas tight, pre-sterilised 50 mL glass vials. The vials were then incubated in the dark and at *in situ* coal seam temperature (\sim 30 °C) for extended period of time.

Coal subsamples produced in this way were then used for two types of microbial gas generation experiments:

- Type 1 or treatment experiments: in these experiments coal subsamples were mixed with a solution of formation water, a reducing agent (to create anaerobic conditions by removing any O₂ in the solution) and a nutrient. The gas evolving from this mixture was then measured after a period of time to quantify the total volume of gas which would be produced by microbial activities but also produced from desorption of any residual gas remaining in the crushed coal.
- Type 2 or control experiments: in these experiments coal was mixed with a biocide solution (usually a solution of 70% ethanol and 30% distilled water) to stop any microbial activities. These experiments were conducted to quantify any gas produced from desorption of residual gas remaining in the crushed coal.

Concurrent treatment and control experiments allows for investigating the effects of microbial activities on the nature of gas evolving from the coal. Molecular and isotopic composition analyses were conducted to characterise gas evolved from these experiments.

COAL SAMPLES

The surface exploration borehole drilled for this study intercepted five coal seams in a coal sequence between depths of 68 and 206 m. Proximate and petrology analyses of all samples were also undertaken to characterise coals for their type and maturity. Petrology results show that vitrinite reflectance (VR) for these samples range between 0.64 to 0.70% and hence these coals can be considered high volatile bituminous rank. The vitrinite content of the samples varied from 50 to 70% on a mineral-free basis. Table 1 reports some of the petrology data and proximate analysis data of coal core samples collected for this study.

Proximate analysis (w/w %)								
Coal seam	Depth (m)	Moisture	Volatile matter	Fixed carbon	Ash yield	Volatile daf	free vitrinite (%)	Vitrinite reflectance (%)
Seam1	68	2.0	35.0	43.6	19.4	44.5	66.9	0.65
Seam2	115	2.1	32.7	44.4	20.8	42.4	59.0	0.64
Seam3	124	2.4	28.2	46.0	23.4	38.0	59.1	0.69
Seam4	160	2.8	27.3	57.4	12.5	32.2	70.3	0.68
Seam5	206	2.6	27.4	58.5	11.5	31.9	50.0	0.70

Table 1: Petrology and proximate analyses of coal samples used for experiments

EXPERIMENTS

As explained in the methodology section, treatment and control subsamples were prepared for coals from five seams. For each coal sample, a total of 12 treatment and 12 control sub-samples were prepared resulting in a total of 120 sub-samples for the five seams studied. It was planned to have three harvests (sampling periods for analysis) for the life of the project spaced about four to five months apart.

For each coal, at the completion of each successive harvest, four treatment and four control subsamples were analysed. These replicates were set up to allow for statistical analysis of the results. The remaining subsamples were kept in the incubator for the following harvests. All samples were prepared in a He atmosphere (with no O_2 present in the system). Treatment and control samples were placed in gas tight glass vials and incubated in a pre-designed incubator at *in situ* conditions.

At each harvest the gas evolved in each experimental vial was measured for the produced volume as well as for the molecular and isotopic composition of the gas. Isotopic composition data are not reported in this paper.

EVALUATION OF GAS GENERATED IN COAL DUE TO MICROBIAL ACTIVITIES

To verify whether microbial activities had taken place and biogenic gas had been produced, the concentration and volume of CH_4 in the headspace of all treatment and control vials were measured. Note that treatment experiments gave an indication of the total gas evolved from a mixture of coal and solutions in the vials. To estimate the volume of CH_4 produced by microbial activities alone, the volume of gas in treatment vials was reduced by the volume of CH_4 in control vials.

The CH₄ production data, in terms of net biogenic gas generated from harvests, are presented in Table 2. Note that the amounts of gas produced during harvests are presented in terms of volume of gas per unit mass of coal. As the amount of gas in coal is commonly expressed in cubic meters per tonne(m^3/t). This unit was used to quantify the volume of biogenic gas production in these experiments. Note that the volume of gas is estimated by measuring the concentration of gas in each experimental vial and the void volume of that vial.

Coal seam	Net bioge	enic CH ₄ produ	$ced (m^3/t)$
	Harvest 1	Harvest 2	Harvest 3
Seam1	1.32	1.56	5.11
Seam2	0.03	0.04	0.12
Seam3	0.06	0.05	0.22
Seam4	0.11	0.10	0.11
Seam5	0.55	0.75	10.52

Table 2: CH₄ produced from microbial activities from coals used in this study

The results show that gas was produced for all coals from all five seams and in all three harvests. The data also show that, with the exception of Seam 4, more gas was produced for the longer harvest periods. The most microbial gas was generally produced in the third or longest harvest (about a year). An interesting observation is that the volume of gas produced does not depend on the seam depth. In fact the coal samples from the shallowest and deepest coals (Seam1 and Seam5) produced the most gas. A maximum of 10.5 m³/t was produced by coals from Seam 5 and about half of this amount (5.1 m³/t) was produced by coals from Seam1. The other three seams produced much smaller volumes of gas generally about or below 0.1 m³/t. Figure 1 shows a plot of the amount of CH₄ produced for each of the five seams for the three harvests.



Figure 1: Net volume of CH₄ produced from microbial activities for the coals studied

Using available petrology and coal quality data for these coals (Table 1), relationships were established between the amount of microbially produced CH_4 and these parameters. However, the petrology data (such as vitrinite content and reflectance) and coal quality data (such as ash yield, moisture content and volatile matter content) do not show large variations across these coals. Therefore, no trends could be established between these properties and average CH_4 production from microbial activities.

CONCLUSION

The analysis of CH_4 gas produced from microbial culture experiments on coals from the sequence of coal seams at a mining zone in New South Wales suggests that CH_4 has been produced for all the seams although at very different rates. The shallowest and deepest coals produced the most gas with about 5.1 and 10.5 m³/t for the longest harvests (~one year), respectively. Other coals from the three remaining seams in the sequence produced small amounts of gas (~0.1 m³/t or less). Overall the results show that microbial activities produce CH_4 in these shallow coals, however, large variations exist between the rates of gas generation for the coals studied.

ACKNOWLEDGEMENT

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ESTIMATE OF THE OPTIMUM HORIZONTAL WELL DEPTH FOR GAS DRAINAGE USING A NUMERICAL METHOD IN THE TABAS COAL MINE

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ABSTRACT: One of the hazards in underground coal mining operations is the sudden coal gas emission leading to coal outburst. To reduce the risk of gas emissions to enable safer mining, it is necessary to pre-drain coal seams and surrounding strata. According to the experimental data, there exists a relationship between the gas flow from the coal seam and the stress changes in the upper layers above the coal seam. This is achieved by drilling horizontal drainage holes in the coal seam. Phase2 commercial software was used to investigate induced stresses caused by methane drainage operations during mining. The optimum depth of horizontal borehole from the coal face was calculated based on the actual gas production. Results of the numerical simulation showed that boreholes with a minimum distance of 30 m from the coalface provide the optimum gas drainage performance for the underground Tabas Coal mine in the South East of Iran.

INTRODUCTION

Methane drainage operation is carried out in underground coal mining to prevent sudden gas and coal outbursts and to enhance safety. Generally, coal beds possess low gas recovery. When the coal face is mined, a pressure difference is generated between the face and somewhere deep inside the coal bed strata, this results in methane emission into the working face. Gas emission is further facilitated by horizontal and vertical fractures induced by the changing ground stress conditions. This paper is aims to study the depth of the coal bed inTabas Mine in Iran, which is undergoing stress variation due to mining activities. As part of this study the following items were investigated:

- Model development to evaluate the increased load on coal bed due to mining activities,
- Investigation of the relationship between stress and the coal gas emission, and
- Development of an incremental approach to evaluate the methane production for various given stress levels. Tabas Coal Mine is located about 60 km South West of Tabas City where the extraction is carried out by longwall mining. The average coal bed gas content is in the order of 15 m³/t.

EVALUATION OF COAL BED BEHAVIOR AND SURROUNDING AREAS DUE TO EXTRACTION

To obtain a reliable numerical simulation, the development of a rock mass model was required. Coal beds differ from other traditional sources for methane emission. Also, they behave differently from other rock masses in terms of mechanical properties.

Phase2 Version 7.0 of Rocscience consulting group, (2010) was used to investigate the rock mechanical behaviour. By using this software, changes in strength properties during loading can be measured separately for the rock matrix and weakening areas. The software allows the selection of the model type (e.g. elastic or elastoplastic behaviour of rock under loading conditions) and various relationships between stress and strain are imbedded in the software as shown in Figure 1.

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Figure 1: Different types of stress-strain curves for different rock behaviors under various loading conditions

The coal bed of Tabas Coal Mine and its overburden were modelled. The model includes a sandstone layer and coal beds. To simplify the model, these layers were considered to be horizontal; however, inclined and oblique seams were created for simulation by changing the boundary conditions or reconstructing the model. The required mechanical properties (bulk modulus, shear modulus, density, cohesion, friction angle, dilation angle, and tensile strength) and pre-fracture changes of different rocks were collected from information contained in reports provided by Tabas coal mine (Tabas Coal Mine report, 1996 and Shereski, 2005)]. The model is 300 m in width and 200 m in height with 1132 elements. Since the extracting coal bed of Tabas is located 200 m underground, a sandstone overburden was considered at the depth of 200 m in the model. Table 1 lists various relevant coal and sandstone roof properties.

Row	Characteristics	Symbol	Val	Lloit	
ROW	Characteristics	Symbol	Coal	Roof	Unit
1	Elastic modulus	E	3	3.5	GPa
2	Poisson's ratio	ν	0.29	0.3	-
3	Tensile strength	σ_t	0.66	2.8	MPa
4	Cohesion	С	0.5	4.7	MPa
5	Internal friction angle	φ	23	32	Degree
6	Density	γ	1600	2600	kg/m ³
7	Volume modulus	K	2.38	2.91	GPa
8	Shear modulus	G	1.16	1.34	GPa
9	Dilation angle	V	10	15	Degree
10	Uniaxial compressive strength	σc	6.62	25.6	MPa

Table 1: Mechanical characteristics of	Tabas coal rock and	sandstone overburden [6]
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As Tabas coal was mined by longwall method, the roof was loaded vertically to simulate the shield supports. Figure 2 shows the designed model in the Phase2 commercial software environment.



Figure 2: Model of Tabas longwall working panel designed in Phase2

MODELLING OF THE COAL MINING PROCESS AND ITS IMPACT ON INDUCED STRESS AND ABUTMENT LOADS

Dimensions of the caved area were modelled. The goaf zone and movement of strata layers were determined as shown in Figures 3 and 4. Because of mining, stresses existing at different points on the model will change, leading to redistribution of stress in the vicinity of the area of extraction. With increased dimensions of the roof area having less support, the collapse of the goaf zone is thus initiated, which extends up to 30 m from the coalface.

ANALYSIS OF MODEL IMPLEMENTATION RESULTS

After the development of the numerical model and examination of the results, the most important matter to be investigated was stress variations within the coal bed and the roof of the coalface. The immediate or nether roof load is naturally transferred on the hydraulic supports and remaining unmined side coal pillars, resulting in increased load on them. These variations in load are presented in Figure 5. It is clears from the figure that the maximum stress on the free surface of the coalface is almost 8.2 MPa. Also, the amount of loading on the goaf zone will initially be zero but starts to increase at a distance of 20 m from the working face. This increase continues and reaches around 6 MPa on the shield supports, and then develops to the coalface. After that, the amount of load on the coal bed starts to decrease and reaches its initial value of 4.4 MPa after about 60 m inbye of the goaf zone. Figure 4 shows the linearly decreasing trend of stress within the coal seam to the depth of 30 m, and then the stress starts decreasing gently. Since at this distance the stress within the coal bed is somewhere between 5.5 and 8.2 MPa, which is greater than the initial stress, the coal bed gas becomes more concentrated. As a result, horizontal drilling causes a greater pressure difference, leading to higher level of gas release from the coal bed. The normal gas content within the Tabas coal bed was obtained as 15 m³/t on average, while the amount of emitted gas from the coalface was measured at around 5 m³/t Tavakkoli, and sereshkil,2006 and Najafi, et al, 2012). The remaining gas difference (10 m³/t) enters the goaf zone through a local fracture, due to the high overburden pressure.



Figure 3: Implementation of designed model and movement of earth layers



Figure 4: Roof collapse and creation of gob zone in Tabas mine working face



Figure 5: Stress variation curve based on distance from gob zone in longwall face of Tabas coal

CONCLUSION

The modeling study demonstrated that there will be a significant improvement in gas capture with the availability of inseam horizontal drill holes close to the working face. The effectiveness of the gas capture in the order of 10 m^3/t may occur as the by horizontal inseam holes becomes within 30 m from the coalface, due to local fractures created by these stress variations. By performing horizontal drill operation and making the required low-pressure space, this gas can be collected and utilised for economic benefit.

The greatest degree of roof stress occurs at the longwall coalface area, and hence requires special attention. The coal bed in this region is compressed and functions as an inflammatory rock mass with increasing coalface stress. This results in the formation of local fractures at the coalface, which increases the risk of sudden gas emission and possible coal and gas outburst.

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REVIEW OF OXYGEN DEFICIENCY REQUIREMENTS FOR GRAHAM'S RATIO

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ABSTRACT: Graham's ratio is a commonly used indicator for measuring the intensity of the oxidation of coal in underground mine atmospheres. Successful measurement of oxygen deficiency is critical in order to generate relevant results, as well as meaningful data trends. Graham's ratio is often used as a trigger for Trigger Action Response Plans (TARP) for the management of spontaneous combustion.

If a Graham's ratio is calculated where there is an insufficient oxygen deficiency the result can be overestimated and trigger a TARP level. Mitchell (1996) and the NSW Mines Rescue gas detection and emergency preparedness book (2014) has previously identified issues with using Graham's Ratio when the oxygen deficiency is less than 0.3%, due to analytical limitations. This issue is often encountered in samples in which the composition is close to air due to the low inherent oxygen deficiency of the sample. The same problem is identified in samples diluted with seam gas. Errors in oxygen deficiency can be compounded by inaccuracies in other measured components when nitrogen is calculated by difference.

A concern with applying the 0.3% oxygen deficiency requirement (minimum limit) to dilute or close to air samples is that valid data may be excluded from interpretation. This paper will review the magnitude and application of minimum oxygen deficiency required for a consistent valid measurement of Graham's ratio. This will be done across a range of samples using real data from a number of modern analysis techniques in underground coal mines.

INTRODUCTION

Graham's ratio is a commonly used gas ratio in the analysis of underground coal mine atmospheres. It is a measure of the efficiency of conversion of oxygen to carbon monoxide. It is also a significant tool in the ability to predict the onset of a heating or the intensity of a heating (Cliff, *et al.*, 2004).

Raw carbon monoxide concentrations are not always indicative of the intensity of a heating due to dilutions or accumulation of gases. By comparing carbon monoxide generated to oxygen deficiency a more relative measurement can be made (Graham's ratio). This measurement is independent of air flow and various forms of the equation account for dilution effects (Cliff, *et al.*, 1999).

The State of Queensland requires continuous gas monitoring and calculation of Graham's ratio in the return for every ventilation split (State of Queensland, 2001). Graham's ratio is often used as a trigger in a coal mines spontaneous combustion Trigger Action Response Plan (TARP) for active longwalls and sealed goafs.

TARP Action levels based on overseas experience are usually set as (Cliff, et al., 2004):

- < 0.4% Normal
- 0.4% to 1.0% Investigate
- 1.0% Heating
- 2.0% Serious Heating / Fire

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However these trigger values may be too high for Australian coal mines (Cliff, *et al.*, 1999) and the following alternative trigger values have been suggested for Australian mines (NSW Mines Rescue, 2014):

- < 0.1%: Normal
- 0.2% to 0.3% Investigate
- 0.4% to 0.7% Heating

Graham's ratio can effectively be simplified to the following equation:

Equation 1:

$$Graham's Ratio = \frac{100 \times carbon \ monoxide \ produced \ (\%)}{oxygen \ deficiency \ (\%)}$$

In order to determine both the carbon monoxide produced and the oxygen deficiency the initial gas readings are essential. The equation 1 is thus expanded to the following form:

Equation 2:

$$Graham's Ratio = \frac{100 \times (carbon \ monoxide_{final} - carbon \ monoxide_{initial})}{oxygen_{initial} - oxygen_{final}}$$

Measurement of the initial carbon monoxide and also the initial oxygen can be done in several different ways. This is mostly dependant on what information is available. If no initial readings are available, fresh air is typically assumed as the initial readings (Cliff, *et al.*, 2004).

The effects of dilution must also be taken into account. When assuming fresh air as the initial gas state, the final nitrogen reading can be used to calculate the initial oxygen result (Cliff, *et al.*, 2004).

Therefore when using an analytical technique that measures nitrogen by difference, the following common form of the equation is derived:

Equation 3:

$$Graham's Ratio = \frac{100 \times (carbon \ monoxide_{final})}{(0.265 \times nitrogen_{final}) - oxygen_{final}}$$

Note that the constant 0.265 is simply the theoretical ratio of oxygen to nitrogen in air. Equation 3 is commonly used to calculate Graham's ratio on real time sensors underground.

Using a measured fresh air value and taking dilution into account is represented by the Equation 4:

Equation 4:

$$Graham's Ratio = \frac{100 \times \left(carbon\ monoxide_{final} * \frac{nitrogen_{final}}{nitrogen_{initial}}\right) - carbon\ monoxide_{initial}}{\left(oxygen_{initial} \times \frac{nitrogen_{final}}{nitrogen_{initial}}\right) - oxygen_{final}}$$

Equation 4 is a common equation used to calculate Graham's ratio for tube bundle monitoring points in underground coal mines. The measured fresh air point is typically from a point on the surface at the tube bundle building, or from an intake roadway underground.

LIMITATIONS DUE TO OXYGEN DEFICIENCY

An oxygen deficiency needs to be present for the Graham's ratio to be calculated. It is impossible for oxygen to be created or generated underground; as such the final oxygen can never exceed the initial oxygen. However, due to the analytical error of analysers and the tolerance for oxygen monitors being \pm 0.2% (Council of Standards Australia, 1990), it is not uncommon for higher final oxygen to be measured relative to the theoretical or measured initial oxygen, even though only a small oxygen deficiency exists. The analyser analytical error is considered normal and expected (Brady, 2007). The example below shows Graham's ratio calculation using a real time sensor gas reading and equation 3:

Gas	Percentage (%)		
Oxygen	20.9		
Methane	0.5		
Carbon dioxide	0.3		
Carbon monoxide	0.01		
Nitrogen (by difference)	78.3		

Table 1:	Real	time	sensor	gas	reading
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 $\textit{Graham's Ratio} = \frac{100 \times (\textit{carbon monoxide_{final}})}{(0.265 \times \textit{nitrogen_{final}}) - \textit{oxygen_{final}}}$

 $Graham's Ratio = \frac{100 \times (0.0001)}{(0.265 \times 78.3) - 20.9}$

 $Graham's Ratio = \frac{0.01}{-0.15}$

Graham's Ratio = -0.06

The negative oxygen deficiency result gives a negative Graham's ratio, which is theoretically impossible and the Graham's ratio result is unusable. A negative Graham's ratio result is an invalid result which cannot be used for interpretation. Real time sensors in return airways often exhibit negative or non-realistic readings of this nature. Figure 1 shows an example of Graham's ratio being calculated from real time sensors in a return airway location with a small oxygen deficiency.



Figure 1: Real time sensor Graham's ratio in return airway location (same data, different y-axis scales)

Figure 1 clearly shows that this Graham's ratio trend is unreliable. Values range from in the negative to in excess of 0.4 which would indicate a heating or fire. This location had a low and constant carbon

monoxide reading, thus it is apparent that the fluctuation in Graham's ratio is mostly due to the oxygen deficiency.

It has been reported that any oxygen deficiency less than 0.2% should be treated with caution due to the possibility of analytical error (NSW Mines Rescue, 2014). Brady (2007) has stated that Graham's ratio can be unreliable for oxygen deficiencies below 0.3%.

By limiting Graham's ratio calculations to an oxygen deficiency of at least 0.3%, which exceeds the allowable error of oxygen sensors in underground coal mines (Council of Standards Australia, 1990), it is far more likely to calculate reliable Graham's ratio. This oxygen deficiency requirement of 0.3% minimum is commonly used as a threshold for the Graham's ratio calculation. Automated monitoring systems are often programmed to remove any readings which don't meet this oxygen deficiency requirement. This eliminates potential unreliable ratio calculations which could contribute to alarm fatigue.

Figure 2 shows the result of Graham's ratio when the oxygen deficiency requirements were applied. The dataset used is the same dataset used in Figure 1.



Figure 2: Graham's ratio with oxygen deficiency \geq of 0.3% oxygen

Note that in Figure 2 most of the Graham ratio results have been eliminated and the data produced is not helpful in generating a trend. This occurrence is frequent for any real time monitoring points with gas atmospheres of low oxygen deficiencies. Brady (2008) concludes: "Due to the variation in measurement and the small oxygen deficiencies present, real time monitoring is not suited to determining Graham's ratio in longwall returns."

Although the occurrence of unreliable Graham's ratio calculations is most prevalent on real-time systems for airway monitoring, it is also possible for tube bundle readings. This is particularly relevant for goaf samples heavily diluted with seam gases with a low oxygen deficiency. This scenario could arise by unintentional air ingress into a sealed area. Because Graham's ratio needs to take dilution into account, errors in nitrogen calculated by difference will result in an error in initial oxygen. Small cumulative errors in other major components thus have an effect on calculated initial oxygen. Gas chromatograph analysis can also experience errors in oxygen to nitrogen ratio due to analytical factors (Brady, 2007). This problem with insufficient oxygen deficiencies due to analytical error exists in all gas analysis techniques.

Current practice

Currently a common practice in automated monitoring systems generating a Graham's ratio is to apply the 0.3% oxygen deficiency requirement to the theoretical oxygen value which will filter out any readings close to air. This eliminates a majority of unreliable data generated in main and return airways. However at the same time this process will indiscriminately filter out useful valid data.

Measurements made with an oxygen deficiency of less than 0.3% may still be reliable in some situations and generate critical data for underground air monitoring.

Unreliable data and false triggers still occur in samples diluted with seam gas because the oxygen is reduced without a significant oxygen deficiency due to dilution. The current practice of basing the oxygen deficiency requirement upon the initial oxygen reading does not help to eliminate unreliable data in samples diluted with seam gas.

METHODOLOGY

Determination of values suitable for testing:

It is believed that the following conditions need to be met for an optimal oxygen deficiency to be determined:

- optimal reduction in unreliable data (avoid alarm fatigue)
- minimal loss of valid data
- increase overall utilisation of Graham's ratio trend and triggers by improving the generation of meaningful data

Two Graham's ratios are presented in Figure 3. One was calculated using a constant carbon monoxide value against an incrementally decreasing oxygen deficiency (triangle) and the second utilizing increasing CO values (diamond).

Figure 3 shows that with both a static or variable carbon monoxide value and an incrementally decreasing oxygen deficiency the trend appears to be linear until around 0.3% oxygen deficiency. Oxygen deficiencies, smaller than 0.05%, give an exponential increases in Graham's ratio with each increment. By inspecting the rate of change for Graham's ratio it is apparent that any Graham's ratio with a corresponding oxygen deficiency below 0.05% will exhibit variability with every increment of oxygen deficiency that it is unusable and thus considered unreliable or invalid. Further observations of Graham's ratio show that, regardless of the magnitude of the carbon monoxide reading, the rate of change of the Graham's ratio trend remains unchanged with decreasing oxygen deficiency as shown in Figure 3.



Figure 3: Oxygen Deficiency against Graham's ratio at a constant CO value (8 ppm) and variable CO

Considering the observed rate of change, the following minimum oxygen deficiency values were used for testing:

• 0.05% – This value appears to be the lowest and most conservative value, as all lower oxygen deficiencies are excessive compared to the previous trend.

- 0.1% This value was selected as a more conservative value, considering data retention as a key requirement.
- 0.3% This is the current literature value; it is the minimum oxygen value to be considered to avoid potential alarm fatigue.

Data collection and processing:

Data in the form of tube bundle and real time monitor logs were obtained from gas monitoring software. These logs were obtained, with permission, from three underground coal mines in Australia, all of which had previously experienced and flagged invalid Graham's ratio triggers in their alarm logs. The locations containing low oxygen deficiencies (around 0.5 or less) were chosen for the study. Each relevant data log was extracted to a Comma Separated Values (CSV) file containing the following information:

- Date and time of measurement
- Monitoring point number (location)
- Methane concentration (%)
- Carbon Monoxide concentration (%)
- Oxygen concentration (%)
- Carbon Dioxide concentration (%)
- Carbon Monoxide Make (Litres per minute)
- Graham's ratio calculated.

These measured gas components are common to all tube bundle and real time analysis used in the testing. Nitrogen is determined by difference in all cases, as the sum of all measured components subtracted from 100%. These default gas components are sufficient to calculate a theoretical Graham's ratio based on theoretical air for the initial values.

In addition to these gas components, the Graham's ratio calculated from the gas monitoring software, as per industry standards, was extracted with each set of gas readings. The CO make value correlating with each data measurement was also extracted where possible.

These extracted data logs were processed in order to calculate a theoretical oxygen deficiency and theoretical Graham's ratio values based on fresh air as the initial readings.

For several tube bundle locations the measured initial air values were used rather than the theoretical initial values. This allowed the Graham's ratio calculation to be replicated as accurately as possible, reproducing the actual values calculated by the mine site monitoring system before extraction. This was not practical to do for all tube bundle locations due to limitations in data processing. Locations processed in this regard were compared to locations processed using theoretical air values, as a means to validate extrapolation of the theoretical data. Real time Graham's ratio was only calculated using theoretical air. The calculated Graham's ratio value for each measurement was categorised based on the following thresholds:

- Normal data was defined as any data with corresponding theoretical Graham's ratio calculated at 0.2 or below. This range is often used as normal conditions for spontaneous combustion management TARPs in Queensland mines (NSW Mines Rescue, 2014).
- Investigate data is defined in this testing as any data with theoretical Graham's ratio calculated at 0.2 to 0.4. This range is often used as an 'investigate' trigger for spontaneous combustion management TARPs in Queensland mines (NSW Mines Rescue, 2014).
- An invalid trigger is defined as any data with theoretical Graham's ratio calculated at over 0.4 without a corresponding significant increase in carbon monoxide or CO make.
• A valid trigger is defined as any data where the theoretical Graham's ratio is calculated at over 0.4 with a corresponding significant increase in raw carbon monoxide or CO make associated with the data. By definition any Graham's ratios over 0.4 which are not valid triggers are considered invalid triggers.

Filtering of minimum oxygen deficiencies

After processing, each set of data was subject to filtering of the measurements based on the corresponding minimum oxygen deficiency being tested. The following information was obtained by comparing filtered data, initial data and reported data from the monitoring system.

- Overall data retention
- Retention of normal data
- Investigate data removed
- Invalid data eliminated
- Valid data eliminated

These values were recorded for each processed dataset and compiled to produce average results for locations of similar type and analytical technique.

RESULTS AND DISCUSSION

Tube bundle data for a three month period at numerous underground locations was processed. The locations investigated were longwall tailgates, goaf seals, main roads and return roads. Measured values were used for the initial gas for some locations. Real time monitoring data for one week was processed from three locations including one longwall tailgate and two main gate locations. All locations were interpreted for valid triggers by identifying any increase in raw carbon monoxide value or carbon monoxide make corresponding with Graham's ratios over 0.4.

Tube bundle: Longwall Tailgate return

The data reported in Table 2 shows that an oxygen deficiency filter of 0.05% or 0.1% retained more data and potential valid triggers than the gas monitoring system reported. The reported gas monitoring data removed 100% of the invalid triggers, however almost 75% of overall data was filtered as well. An oxygen deficiency filters of less than 0.3% and higher than 0.05% appears to be optimal for these locations. A minimal oxygen deficiency of 0.3% was proven to be too high for these locations as the majority of the data was removed, similar to the data reported by gas monitoring software.

Filter	Average Retention of normal data (%)	Average investigate data removed (%)	Average Invalid triggers removed (%)	Valid triggers removed (%)	Total data sets
Gas monitoring system reported	25.8	82.1	100.0	100	2.0
minimum 0.05%	93.6	35.0	95.2	0	2.0
minimum 0.1%	66.3	58.9	96.8	20	2.0
minimum 0.3%	27.7	100.0	100.0	100	2.0
total valid triggers				5	

Table 2 Longwall Tailgate Return - theoretical initial air

Filter	Average Retention of normal data (%)	Average investigate data removed (%)	Average Invalid triggers removed	Valid triggers removed (%)	Total data sets	
Gas monitoring system reported	25.8	87.5	100.0	50.0	2.0	
minimum 0.05%	96.4	38.1	98.7	0.0	2.0	
minimum 0.1%	49.3	82.8	100.0	100.0	2.0	
minimum 0.3%	20.6	100.0	100.0	100.0	2.0	
total valid triggers				2		

Table 3: Longwall Tailgate Return - measured initial air

Tube bundle: Main road and seals

Testing of tube bundle gas monitoring results from both the mains and seals reflected a similar outcome to the longwall tailgate results with a minimum oxygen deficiency of 0.05% appearing to be optimal value for retention of data and elimination of invalid triggers. Application of 0.1% and 0.3% resulted in an unacceptable data loss of valid triggers and overall data. Using a measured initial air value decreased the rate of valid and invalid data.

Real Time: Longwall return and Mains

Similarly to the tube bundle data, 0.05% and 0.1% minimum oxygen deficiency outperformed the measured reported data in terms of data retention (Real Time: Longwall return and mains Table 4). In contrast the minimum oxygen deficiency of 0.05% was not high enough to filter out enough of the invalid readings for practical application. Due to the frequency of real time measurements taken, this would not be sufficient to prevent an excessive number of false alarms. An oxygen deficiency of 0.1% was able to remove all of the invalid triggers while retaining all of the valid triggers and 76.7% of the overall data. It is recommended that an oxygen deficiency filter greater than of 0.05% should be used for real time systems.

Filter	Average Retention of normal data (%)	Average investigate data removed (%)	Average Invalid triggers removed (%)	Valid triggers removed (%)	Total data sets	
Gas monitoring system reported	64.9	65.1	100.0	75.0	3.0	
minimum 0.05%	99.7	21.4	56.7	0.0	3.0	
minimum 0.1%	76.7	36.9	100.0	0.0	3.0	
minimum 0.3%	50.3	75.0	50.0	66.0	3.0	
total valid triggers				12.0		

Table 4: Real time - longwall return and mains

Real time data generated excessive investigate triggers compared to tube bundle results making the technique less suitable for effective trending.

CONCLUSION AND RECOMMENDATION

• Tube bundle data is more effective for trending Graham's ratio than real time.

- A minimal overall oxygen deficiency of less than 0.3% will improve data retention and still have effective removal of invalid triggers for tube bundle calculated Graham's ratio trends compared to the current process of filtering.
- An optimal minimal overall oxygen deficiency for real time calculated Graham's ratio lies between 0.1% and 0.3%. Further investigation to determine optimal value is recommended.
- A minimal oxygen deficiency of 0.3% leads to an indiscriminate loss of potentially valid data for atmospheres close to air, without a decrease in false alarms in atmospheres with low oxygen deficiency.
- Implementation of these optimal tested values should be considered for trial and further review.
- Further testing and investigation should be done to determine an optimal theoretical oxygen deficiency filter.

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ASSESSING THE REACTIVITY OF PYRITE

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ABSTRACT: The presence of pyrite in coal has often been attributed to being a factor in spontaneous combustion, but quantifying the effect has been problematical from a hazard assessment viewpoint. A simplistic approach of correlating high sulphur content with the presence of reactive pyrite is unreliable and can be misleading. Pyrite needs to be present in an appropriate form (size and morphology) to enhance its oxidation potential. Consequently, it is important to be able to identify and quantify any reactive pyrite contribution to coal mine heatings in a systematic and scientific manner. In 1992, the US Bureau of Mines recognised there was a deficiency in existing testing techniques and recommended that these would need to be changed to assess the effect of pyrite in the self-heating process. Recent advances in adiabatic oven testing have led to development of a new spontaneous combustion Incubation Test that is used to benchmark coal self-heating against the known performance of case history coals. This test is capable of measuring the pyrite oxidation reaction for site-specific conditions and not only quantifies the minimum incubation period for pyrite initiated events to develop, but also shows the manner in which they develop.

INTRODUCTION

The US Bureau of Mines conducted an investigation into the cause of floor self-heatings in an underground coal mine operating in a high rank low volatile bituminous coal seam (Miron, Lazzara and Smith, 1992) and concluded that pyrite oxidation was the primary cause of the heatings due to the form (size and morphology) of the pyrite present in the mine floor horizon. The commonly used self-heating indices for the coal indicated a low spontaneous combustion propensity. Consequently, they concluded that there was a need to amend current spontaneous combustion testing methods to fully assess the effect of pyrite on the self-heating process.

Beamish and Beamish (2011 and 2012) performed adiabatic oven tests on a high volatile bituminous coal, which clearly showed the accelerating effects of reactive pyrite on the coal self-heating process. More recently, Beamish and Theiler (2016) have shown that adiabatic oven incubation testing can also be used to assess the self-heating behaviour of reactive pyrite in black shale waste rock from a metalliferous mine. This paper presents the results of adiabatic oven testing of coal samples that contain high sulphur contents to show the success of incubation testing to distinguish between reactive and non-reactive pyrite and to quantify the effect of reactive pyrite on the self-heating process.

Coal Samples

Analytical details of the high volatile bituminous coal samples used in this study are contained in Table 1. Coals C and D are from the same seam and location at an underground mine with a known history for spontaneous combustion events. Both samples have high total pyritic sulphur contents (Table 2) and X-ray Diffraction Analysis has confirmed that the main mineral constituent in the coal is pyrite. Coal C is a hand-picked part of the seam where pyritic bands parallel to bedding are concentrated. These samples also have high organic sulphur content as shown in Table 2.

Coal E is from another underground coal mine that does not have a history of spontaneous combustion events. The sample location is associated with the presence of a clastic dyke that has intruded into the seam from the roof. In this case X-ray Diffraction Analysis has identified the main

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mineral constituent as marcasite (the orthorhombic form of pyrite). This sample also has an elevated organic sulphur content (Table 2).

Sample	Moisture	Ash	Volatile	Calorific	ASTM
	(%, arb)	(%, db)	Matter (%, dmmf)	Value (Btu/lb, mmf)	rank
Coal C	1.1	17.3	43.4	15137	hvAb
Coal D	1.6	8.4	44.8	15372	hvAb
Coal E	2.4	12.2	37.3	14473	hvAb

Sample	Pyritic Sulphur (%, db)	Sulfate Sulphur (%, db)	Organic Sulphur (%, db)	Total Sulphur (%, db)
Coal C	9.70	0.21	3.09	13.00
Coal D	3.32	0.02	2.31	5.65
Coal E	3.23	0.26	1.90	5.39

Table 2: Forms of Sulphur present in high Sulphur coal samples

Spontaneous combustion index parameters

The more commonly used spontaneous combustion index parameters in the Australian Coal Industry are initial self-heating rate (R_{70}), Minimum Self-heating Temperature (SHT), Crossing Point Temperature (CPT) and Relative Ignition Temperature (RIT). The values of these index parameters for each of the high sulphur coal samples are shown in Table 3 and the R_{70} self-heating rate curves for the respective coals are shown in Figure 1. It can be seen that Coals C, D and E all have low spontaneous combustion propensity ratings. The ratings for Coals C and D are completely at odds with the known spontaneous combustion history of the seam.

Table 3: SHT, CP	۲, RIT and R70 values for the coals used in this studywith their respective
	spontaneous combustion propensity ratings

Sample	SHT (⁰C)	СРТ (ºC)	RIT (⁰C)	R ₇₀ (⁰C/h)	Propensity Rating
Coal C	114	157	158	0.29	Low
Coal D	116	159	154	0.06	Low
Coal E	104	148	156	0.48	Low





The R_{70} self-heating rates of all these samples are much lower than normally recorded for their respective rank and ash content. This suppression in reactivity has been observed to correspond with coals that contain high organic sulphur content. It is likely that the organic sulphur may be acting as a natural inhibitor to oxygen reaching reactive sites.

Incubation testing quantification of reactive/unreactive pyrite

The commonly used spontaneous combustion indices only provide a rating of the intrinsic reactivity of the coal. They do not show moisture moderating effects of the self-heating rate at low temperatures, nor do they show the mutual moisture/pyrite effects identified by Arisoy and Beamish (2015a). One of the essential ingredients for pyrite oxidation is water. The R_{70} test (Figure 1), which is conducted on a dry basis, records only the intrinsic coal self-heating and no pyrite self-heating. However, the incubation test of Coal C for example (Figure 2), which starts at a lower temperature and with moisture present, rapidly self-heats and the test is completed in approximately one quarter of the time.

The self-heating rate between 40 and 70 °C in the incubation test is 5.12 °C/h compared to 0.29 °C/h for the R_{70} test. Also, the shape of the self-heating curve is different for the pyritic reaction at low temperature. The non-Arrhenius kinetics that occurs for the coal oxidation reaction (Arisoy and Beamish, 2015b), where the coal self-heating rate decreases before increasing again at higher temperatures due to reactive site availability, is not present for the pyrite oxidation reaction contribution. Therefore, these results clearly show the reactive nature of the pyrite present in this sample and the mutual effects of pyrite and moisture on accelerating the overall coal self-heating rate from the initial start temperature.



Figure 2: Comparison between incubation test results and R₇₀ test results for coal C

Clearly, the moisture available for the pyrite oxidation reaction is a key factor in the overall spontaneous combustion propensity of coals containing reactive pyrite. This is shown in Figure 3 for repeat tests of Coal D at different moisture contents. At low moisture levels there is insufficient water available for the pyrite oxidation to proceed at appreciable rates. At high moisture levels there is too much water available and the self-heating is hindered by the heat loss from moisture evaporation. This is clearly visible in the temperature region of approximately 95 °C where a significant inflection appears in the self-heating curve.

Coal E initially self-heats at a very slow rate (Figure 3), and shows no accelerated self-heating with time, indicating the marcasite that is present in the coal is unreactive. This is at odds with the existing literature, which suggests that marcasite is a more reactive mineral than pyrite under oxidation conditions. These new results would indicate that previous studies have not taken into consideration the influence of the size and morphology of the marcasite on the oxidation reactivity.



Figure 3: Incubation test results for high sulphur coals showing both reactive and unreactive pyrite self-heating performance, including the effects of moisture availability

CONCLUSION

Identification and quantification of the presence of reactive pyrite in underground coal mines is crucial to developing an appropriate Principal Mining Hazard Management Plan for Spontaneous Combustion. Commonly used index parameters do not provide this information and can be misleading in some instances, particularly for higher rank low reactivity coals and shale units present in immediate roof and floor rocks. Moisture availability plays a major role in the pyrite oxidation reaction and this can be easily demonstrated by using the incubation test method that has recently been developed. The test provides a realistic record of the characteristic self-heating performance of the sample being tested and produces an indication of the minimum incubation period for a spontaneous combustion event to develop. This information is invaluable for safe and effective mining of coal seams where reactive pyrite may be a contributing factor to the spontaneous combustion hazard.

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HOW RELEVANT ARE ENGINEERING SAMPLES IN THE MANAGEMENT OF PERSONAL DUST EXPOSURE?

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ABSTRACT: A directive, legislated by the South African Department of Minerals and Energy (DME) in 1997, was introduced to reduce the respirable dust exposure of Mechanical Miner (MM) operators to below 5 mg/m³, when measured at the operator's cab position. This was to be achieved by ensuring that ventilation and dust control systems are effective in minimising the worker dust exposure. The focus of this paper is to review the effectiveness of this rule for almost two decades by using engineering sampling data to compare cost of monitoring versus success in dust control, to discuss perceptions arising from the application of this rule and to suggest improvement opportunities in the management of this hazard within the South African industry. The results of this study have demonstrated that the fixed-location Continuous Miner (CM) engineering sample results cannot predict the shift dust exposure of an MM operator. Therefore, it is recommended that the CM engineering sampling, as currently practiced, should be reviewed with the potential objective to discontinue it and replace it by personal exposure monitoring using the new MSHA-approved realtime monitoring device (NIOSH PDM3700). This instrument is able to collect relevant engineering dust control data for effective exposure management. The conclusions of this paper are based on extensive data analyses and should enable each mine and the regulator to make step-changes in current daily engineering sampling requirements and provide the flexibility required to approach the management of personal exposure more effectively by reducing human errors in sampling and optimising the use of available resources for the benefit of the South African Mining Industry.

INTRODUCTION

Major hazards in an underground coal mine include methane and coal dust explosions and personal exposure to dust. Based on the first ever recorded coal mine explosion in Southern Africa in Natal in 1891 (Landman, 1992), it can be inferred that underground coal mines have been operational for over 125 years. Considering where the global coal mining industry is positioned today and the arduous efforts that have resulted in improved public perceptions of the coal industry, Figure 1 demonstrates the success of various initiatives in reducing fatalities resulting from underground gas and dust explosions.

The statistics shown in Figure 1 include events from early 1900s to 2015 from USA, South African and Australian coal mines. Coal mining in South Africa has matured over the decades in both safety and health management with a unified approach towards management of risks. The reduction in explosions is factual evidence of the coal mining industry taking responsibility and being proactive in preventing such major incidents with innovative technologies, technical leadership and being eternally vigilant in dealing with multiple hazards in the workplace.

Understanding the risk of exposure to respirable dust was pioneered in South Africa in the early part of the 20th century with initial dust sampling techniques employing the konimeter, the use of the realtime Hund Tyndallometer, and later the introduction of gravimetric sampling in the mid-1990s, despite the USA and the rest of the world adopting it in the early 1970s. Similarly, the management of exposure to coal dust has also improved over the last two decades with major industry initiatives and regulatory interventions. One of the milestones in dust management in coal mines was a directive,

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effectively termed as "the 12 m rule", introduced by the South African Department of Minerals and Energy (DME) in 1997. In addition, during this period, South Africa became the first country in the world to adopt the new ISO/CEN/ACGIH size-selective respirable dust curve for monitoring dust as opposed to the original Johannesburg size-selective curve of the 1960s (Orenstein, 1960; Belle, 2004).

Increasing concern about coal dust related lung diseases, together with the Leon Commission Report (1995), caused the South African DME to review the legislation aimed at protecting the health and safety of mine employees. Directive B7, titled "A Guideline for the Ventilating of Mechanical Miner Sections" was issued by the DME to the South African coal mining industry. This directive stipulated that one daily dust sample, termed "a CM engineering sample" was to be taken at every Mechanical Miner with an acceptable limit of 5 mg/m³. The sampling pumps were to be positioned on the CM at the operator's position or at a position where the CM operator would be seated if on board the machine. Analysis of the results (Belle and Phillips, 2003) indicated that mere application of 12 m rule on its own does not solve the dust problems, but rather that this is achieved by the meticulous application of available state-of-the-art dust control technologies, leading work practices, and the regular maintenance of installed systems to ensure that they work at all times. In combination these measures would ensure a reduction in worker exposure to coal dust.



Figure 1: USA-SA-Australian coal mine explosion fatality statistics over the decades

The 12 m rule directive for dust exposure management states as follows (DME Directive, 1997):

- 1. "No continuous miner (CM) heading must be developed further than 12m from the last row of permanent support or from the point of auxiliary ventilation; and
- 2. Only ventilation systems that can ensure, at all times, a maximum dust reading of 5 mg/m³, measured at the driver's position on the continuous miner (CM), must be employed."

This paper reviews the origin of the CM engineering sample, its application and shortcomings and its current interpretation after two decades of implementation in South African coal mines.

Dust monitoring in South African Collieries

This section of the paper summarises the history of dust exposure monitoring and the changes that have taken place in the last two decades. Exposure monitoring and assessment is a complex system that requires clear understanding of the coal mining operation, monitoring practices, engineering

controls, ventilation system and dust generation dynamics. It is therefore increasingly necessary to measure the dust levels as accurately as practicable to assess the exposure, by using effective sampling techniques. Historically, the assessment of workers' dust exposure in South African coal mines was done by using various air samplers such as the Casella 10 mm cyclone, Gilian cyclones, GME008 Higgins-Dewell type South African cyclones, MSA cyclones, and CIP10 samplers. All these dust monitoring units were approved by the DME and operated at a conventional flow rate of 1.9 L/min, except for CIP-10, where the flow rate is 10.0 L/min. Due to its inherent measurement shortcomings (Belle, 2002), CIP10 samplers are no longer used in South African mines following an instruction by the DME. Currently South African coal mines must perform two types of dust sampling. In terms of the DME guideline for the assessment of personal exposure to airborne pollutants (August 2002), the results of the personal exposure sampling programme are to be submitted to the inspectorate quarterly. In terms of the Department of Minerals and Energy Affairs Guideline for a Code of Practice for the Ventilating of Mechanical Miner Sections in Coal Mines in terms of Section 34(1) of the Minerals Act 1991 (Reference GME 16/2/1/20 dated October 1994), also known as "Directive B7" or the "12 m rule", the results of gravimetric sampling performed daily at all operating CM sites, termed "environmental samples" in the directive, but commonly referred to as 'engineering sampling' must be submitted to the Directorate within four days.

Prior to 1998, dust samplers at all South African underground mines were operated at a flow rate of 1.9 L/min in agreement with the BMRC respirable convention (BMRC, 1952). However, according to the new ISO/CEN/ACGIH respirable dust curve with a 50% cut point (d_{50}) of 4 µm, the recommended flow rate is 2.2 L/min (Kenny, Baldwin and Maynard, 1998). Currently, mine dust sampler pumps draw 2.2 L/min of air through a mini-cyclone, which separates the airborne dust and collects only the fraction of respirable dust (<10 µm) on a pre-weighed filter. The dust samples are weighed on completion of the working shift and the procedure for determining the dust mass is followed according to DME guidelines (1995).

BACKGROUND TO ENGINEERING SAMPLING

This section of the paper provides background to various sampling definitions that are used in the mining industry. Occupational health exposure assessment refers to various sampling strategies over the years and relevant definitions of the sampling methods are summarized (Belle and Clapham, 2001) below:

Personal sampling: Is a method of sample collection whereby the dust sample collected is in the breathing zone of a mine worker while performing occupational duties during a work shift. In this sampling method, the worker wears the sampling train (cyclone, pump, tube, sample filter) for the entire work shift. Personal sampling results are most commonly used as the exposure or dose element in the development of dose-response relationships.

Area or environmental sampling: Is a method of sample collection whereby the dust sample taken at a fixed location at the workplace in an environment or area of interest that is not mobile. The dust sample reflects the average concentration in the area of interest and does not reflect the exposure of any worker in that area. The guideline for a code of practice for the ventilating of mechanical miner sections in coal mines (1994) noted that the sampler is to be placed in a stationary position inside the cab of the mechanical miner and referred to as environmental sampling. Area sampling should not be confused with the engineering sampling suggested in the Directive (1997) and the term "environmental" for the purpose of B7 is not correct.

Occupational sampling: An occupational sample is the dust sample taken during a work shift on individual workers who perform duties in a designated occupation and the terminology is used in US coal mines. This method of sampling measures the dust exposure for defined occupations as if one person performed the duties in that occupation for the whole working shift.

Engineering sampling: An engineering sample is the dust sample taken at the CM operator's position, which is not defined in the original DME directive (1997). The origin of the engineering sampling in South Africa can be traced back to a B7 directive (1997), requiring underground coal mines to reduce the dust levels to below 5 mg/m³ for the sampling period at the operator's cab position on CMs. An engineering sample is the dust sample taken to characterise the emission source or suppression effectiveness of ventilation and dust control measures. The engineering sampler is switched on at the face area at the beginning of the shift while the cutting machine is standing and is switched off before leaving the face area at the end of the shift. It aims at evaluating both the management (administrative effectiveness) of the dust control system as well as effectiveness of the dust control system (engineering). An engineering sample (sample collected during the sampling period only) is the dust sample taken at the CM operator's position (Figure 2). The engineering sample is collected only while the engineering activity is taking place (in this case CM operation).



Figure 2: Position of samplers at the CM operator's position

What is of importance in the current context is that when the directive was instituted and promulgated during the late 1990s, the CM operator was on-board the machine. Currently, the majority or almost 90% of CM operations are done remotely where the operator is in fresh intake air. In addition, there was no guidance, in the B7 directive on the exact location of sampling with respect of the CM cab geometry other than "front of the CM cabin", as should be specified in evaluations of various engineering dust control systems.

Figure 3 shows the position of the instrument used to obtain a CM engineering sample, (i.e., location '1' in Figure 3) as per directive B7 that applied in Mine Health and Safety Council (MHSC) studies (Belle and Du Plessis, 1998). The choice of location-1 provides an indication of dust roll-back at the CM operator's position and the effectiveness of directional sprays and the ventilation system and of the CM dust control system when the CM is operated with an on-board scrubber and auxiliary ventilation system. Operating CMs 'remotely' as is common now allows the CM operator to be located in the fresh intake air (location R in Figure 3). With the switch over to remote operation and the operation of larger CMs, the position of the sampling device was also moved away to other locations towards the back of the CM (Locations 3, 4 and 5 in Figure 3). This has resulted in failure to adhere to the DME guideline (1994/1997) regarding compliance test requirements and has invalidated any comparison of the results obtained over the years. In some instances, a dust pump steel mesh box was built with engineering samples positioned inside it. During another dust control system study, two real-time dust samples were taken in front of the CM cabin (location 1 and 2). Results from these studies shown in Figure 4 highlight the difference in the dust cloud sampled by the monitors. This demonstrates the dynamics of the sampled dust cloud, in particular, the peak dust levels between the two real-time monitors when evaluating the effectiveness of the control system in managing the methane and dust hazards and the importance of the location of the CM engineering sample monitoring station.



Figure 3: Position of samplers at the CM operator's position-Circle 1 (Source: Belle and Du Plessis, 1998)



Figure 4: Real-time dust levels monitored for sample locations (1 and 2) identified in Figure 3

In the above example, despite the dust samplers being located approximately 60 cm from each other, the dust cloud monitored by the instruments was different. Such variability in measured peak dust levels shows the complexity and doubtful validity of any conclusions that may be drawn from this sampling. In this specific example, the engineering sample dust level was 4.75 mg/m³ (average of peak flammable gas level was 0.12 %) with a shift production of 1020 tons. If the CM was to be operated under remote control, the CM operator would be standing in the fresh intake air (dust concentration measured at 0.29 mg/m³). The position of the dust samplers used in the engineering area sampling is crucial in determining the effectiveness of dust control systems. This can easily be illustrated by positioning a sampler at different locations, e.g. closer to the flight conveyor, CM cutter head. Comparisons between dust sampling results therefore require consistent positioning of the samplers. Further examination of the concentration-time data from underground trials shows how real-time dust sampling instruments placed at a distance of less than a meter from each other in the CM operator's cabin actually monitor two different dust streams (Figure 4). In this example, Hund-2 was positioned inside the operator cabin, towards the CM flight conveyor, while Hund-1 was

positioned inside the cabin towards the clean air side. The conversion factor for the Hund data was obtained from the gravimetric samples collected at the Hund-1 position. The average concentration levels from the two Hunds for the total sampling period were expected to be similar. Further analysis of the Hund data for a specific cutting period shows that the measured dust exposure levels differed as the monitors sampled two different dust streams near the CM cabin (Table 1). This clearly illustrates that a face worker can be exposed to different dust concentration clouds and refutes the view that the dust exposure level within even a small area is fixed. This illustration gives an idea of the complex nature of sampling, analysis and interpretation of the dust concentration values obtained in the field.

Specific Cutting Period	CM Operator Concentration Level, mg/m ³				
Specific Cutting Period	Hund-1	Hund-2			
Period 1	5.38	6.37			
Period 2	4.99	4.88			
Period 3	6.70	6.43			
Period 4	5.84	6.68			

Table 1: Concentration levels at the CM operator

Ultimately, the above results give an idea of the serious difficulties of dust measurement, with many challenges in obtaining the consistency required when comparing with a set limit for effectiveness of controls.

Typically, engineering samples are used to identify failures of engineering controls and such sampling is not a common practice elsewhere in the world for routine and daily sample data collection as practiced in South Africa. In order to compare the concentration of dust measured for similar CM operations data from a development heading (Figure 5) in a US coal mine is used. Dust concentrations were measured at the left and right rear corners of the continuous miner and at the remote miner operator location, with and without the additional side sprays in operation (Goodman, 2000). The results clearly show the significant differences in the measured dust concentrations onboard the CMs and for the CM operator at the remote position. Regardless of the dust control system operation (on or off), the ratio of fixed location area sampling to CM remote operator was high, i.e., 3.69:1 (sprays off) and 4.30:1 (sprays on) referenced to the left rear corner sampling position. Table 2 highlights the engineering dust concentrations greater than 5 mg/m³ measured by this NIOSH research study (Goodman, 2000).





Dust Control	Sample Location	Number of Samples, #	Sample time, minutes	Average dust, mg/m3	Fixed/Remote CM Ratio, #
Side sprays Off	Left rear corner	2	321	9.74	3.69
	Right rear corner	2	321	4.58	1.73
	Machine operator	2	103	2.64	1.00
Side sprays On	Left rear corner	2	338	8	4.30
	Right rear corner	2	340	4.4	2.37
	Machine operator	2	120	1.86	1.00

Table 2: Relationship between CM mounted fixed sample and remote operator from a US coal mine (Goodman, 2000).

Myths about CM engineering sample results as coal dust explosion diagnostic tool

Over the years, there have been various misleading views on the value of collecting engineering samples daily and of comparison with the stated limit value of 5 mg/m³. One such view is that 5 mg/m³ limit value can be a diagnostic metric to assess the coal dust explosion risk. The author has attempted to clarify this claim in the context of minimum dust levels required to initiate dust explosions. Coal dust is both a health and an explosion hazard and the key differentiator between these is particle size. Coal dust is less homogeneous compared to industrial dusts and has a complex structure which affects the propagation of a coal dust explosion. For health risk assessments, mean particle sizes of 4 μ m (D₅₀ value) is of significance with the current coal dust exposure limit of 2 mg/m³. On the other hand, for dust explosion risk assessment, mean particle sizes of 20 μ m (D₅₀ value) are used at least in explosion simulation test facilities such as Kloppersbos. There are various values on minimum dust explosion initiation limit values that can be found in the literature based on type of ignition source used, and laboratory limitations. One of the known lowest dust explosion initiation concentration value is 37 g/m³ or 37000 mg/m³. In addition, the commonly used Kex value to determine the explosibility of coal dust at Kloppersbos uses the dust level of 500 g/m³ or 500,000 mg/m³. Based on this critical and irrefutable scientific evidence, it is to be noted that the current B7 'engineering CM sample' value of 5 mg/m³ cannot be used as an input to assess the coal dust explosion risk assessment.

The levels of respirable and explosive dust formed during the coal mining process will depend on the inherent dustiness of the coal seam, pre-drainage of coal seams for methane, the array of cutting tools used and the design of the mining machine used. It is not easy to find documentary evidence of measured dust levels at the coal face of mines globally. This is due to the challenges of collecting samples or the conditions. In the dust explosion research work by Landman (1992), it was documented that the dust levels measured by Kachan, Kocherga and Kolchinsk in the Donbass basin coal mines in the Ukraine were recorded as between 4.6 g/m³ to 12.3 g/m³ at a distance of 0.5 m from the coal face, with 19.8 g/m³ to 25.8 g/m³ being the maximum. With water sprays the dust level around the cutting pick remains high at 9.2 g/m³, but falls to 0.4 g/m³ to 3.6 g/m³ when 0.5 m from the face. However, there was no mention of the sampling techniques used in the study. Based on dust measurement experiences at various coal faces, it is considered that these dust value can be very subjective, especially when taking samples while the CM is cutting with dust control systems operational, i.e., sprays, scrubbers.

Landman (1992) also reported gravimetric samples of the dust clouds surrounding CM drums measured by Chamber of Mines research Organisation (COMRO) from various collieries in South Africa. The recorded values varied substantially, with mean particle sizes of between 20 and 50 microns. It is to be noted that the dust concentration values are to be used just as a guidance and not as an absolute figure as the sampling during coal cutting is very unsafe and complex and values can vary enormously due to the type of dust gradient used from coal face to outbye area. The dust

concentrations observed varied substantially in a wide band from 1.7 g/m³ to a 160 g/m³, but values could possibly be higher. Landman (1992) has reported that high dust concentrations with methane far below its lower explosive limits can result in hybrid mixtures that are extremely sensitive to ignition. The most explosible mixtures for methane occur at the stoichiometrically balanced point of 9.5 % methane, and for coal dust depends on the properties of the coal under investigation. For washed Ermelo coal dust mixed with air, stoichiometric balance is achieved at 625 g/m³ of coal dust, for unwashed Ermelo coal at 670 g/m³, for washed Springfield coal at 928 g/m³ and for un-washed Springfield coal at 1033 g/m³.

For point source ignition, the lower explosive limit of a methane/air mixture is 5.2 %, but 1.4 % is the maximum allowed by law, resulting in a methane safety factor of 3.71. Similarly, if the lower explosive limit of Ermelo dust for volumetric chemical ignition is 75 g/m³ (Landman, 1992), limiting concentrations to about 20 g/m³ will also result in a safety factor 3.75. If the engineering CM dust value is seen as the limiting dust level for explosion prevention, the safety factor would be 15,000. For hybrid mixtures, which can be defined as a combination of two different phase components, namely gas and dust (solid) material in air, Bartknecht (1987) reported that at 2% methane concentration, the Lower Explosive Limit (LEL) of coal dust drops by 64%. Therefore, the safety factor in this discussion would drop down to 9,600 from 15,000.

The information evidenced above clearly show that the engineering CM dust concentration limit of 5 mg/m^3 is well below the minimum dust explosibility level needed to initiate dust explosion in a coal face and therefore the use of this limit as a proxy measure for dust explosion risk is not deemed feasible.

RELATIONSHIP BETWEEN PERSONAL VS CM ENGINEERING SAMPLE DATA ANALYSES-DISCUSSIONS

The consistent application of the Directive (1997) represents almost two decades' worth of engineering sample collection by all underground coal mines in South Africa. In an effort to provide some relevance to this work, a pairwise analysis of the engineering and personal CM operator sample data was carried out in order to evaluate the relationship between CM engineering and CM operator sample results. The dust levels presented throughout this paper reflect respirable gravimetric dust measurements taken over a full working production period. Although the original engineering sampling definition was meant for the production period only, some of the engineering sampling periods were greater than 8 hours. It was assumed that the dust samples as collected underground were weighed and the procedure for determining the particulate mass was followed according to DME guidelines (DME, 1995). A total of 200 pairwise samples were available from two differences in findings.

Figure 6 provides the relationship between engineering and personal CM operator sample values for different periods in two decades of sampling. The poor relationship between the engineering and personal CM operator samples is evident from these results. Similarly, Figure 7 shows the frequency distribution of the ratio between CM engineering and personal samples for the two sampling periods in the last two decades. It was noted that in the pre-2010 era, nearly 43% of the samples exceeded the ratio of 5:1, while post 2010, 22% of the samples exceeded this value. The key reasons for the extreme concentration ratios and significant differences in engineering and personal exposure levels can be attributed to the following:

- The outbye location of CM engineering samples (Figure 3) and off-board CM operator and different size of CM types impacting the dust gradient from the coal face and the engineering sample location.
- Human errors associated with the engineering and personal sampling process underground.
- Impact of stone dusting during engineering and personal sampling significantly affecting the interpretation of the measured dust levels as identified by Belle and Phillips (2013).

• Improved ventilation system and scrubber operation to improve the efficiency of the dust sizecapture profile resulting in reduced dust levels

During a number of field studies, the following observations were made with regard to misuse of samplers. The problems observed include: incorrect sample pump and cyclone handling procedures, inconsistent positioning of samplers on the machine, sample pump not switched on, pump operating at flow rates of 1.4 l/min or below, sample pump pipe not connected properly, coarse dust holder (cap on bottom of cyclone) missing, sample pumps forgotten in lamp room, waiting place, on the top of auxiliary ventilation devices like a force fan and jet fan, lack of discipline and little or no knowledge of operating procedures and reason for sampling and/or benefit to operator not being clear. During frequent visits underground at a number of mines, it was found that the pumps were not switched on – this could have been the result of either pump failure or poor discipline.

As noted from the regression equation with a very poor correlation for both of the two periods, for a 2 mg/m³ personal Occupational Exposure Limit (OEL), the resulting engineering CM dust sample value would be 1.37 mg/m³ and 1.35 mg/m³ respectively against the expected CM engineering sample Directive (1997) value of 5 mg/m³. This further demonstrates the failure of the original rationale on the derivation of the 5 mg/m³ engineering CM dust limit value as an indication of personal exposure level of CM operator or face area workers.



Figure 6: Relationship between CM engineering and personal samples



Figure 7: Frequency distribution of ratio of CM engineering and personal samples

USE OF CM ENGINEERING SAMPLING FOR PERSONAL EXPOSURE ASSESSMENT

Various studies have indicated that personal sampling provides the best estimate of worker exposures and of the temporal and spatial variability in those exposures for use in dose-response models. Leidel *et al.* (1977) recommended that, for accurate assessments, the personal exposure measurements must be taken within the worker's breathing zone. The inaccuracy incurred in using area sampling for measuring dust exposure of mining machine operators in US coal mines is well documented by Kissell and Sacks (2002). They recommended that the worker exposure is best assessed using 'personal sampling' rather than 'area or engineering sampling' techniques using evidence based on US coal mine studies. However, experience of personal and area dust

measurement in the narrow reef and humid conditions of very deep gold mines in South Africa suggests that area sampling can be an option where a clear dust gradient can be established with minimum variability between the two sampling techniques (Belle, 2002). The following studies reflect the extent of and the possible reasons identified for using personal sampling for exposure assessment:

- A comparative study of personal and fixed-point (area) samplers by Breslin, Page and Jankowski (1983) reported the coefficient of variation of measured mine dust concentration to be typically less than 20%.
- Listak *et al.* (1999) concluded that there was little predictive correlation between fixed-location area samples on CMs to operator breathing zone samples. This US study noted that if the fixed-point dust level was 1.5 mg/m³, then the 95% confidence level predicted operator dust levels at the boom hinge point and in the operator breathing zone exposure could vary from zero to 2.6 mg/m³.
- Divers *et al.* (1982) conducted a three-shift dust study in a US coal mine operated using remote control machines. Their study showed that the mean ratio of respirable dust samples taken at the cab and at the remote control operator location was 30.7.
- Kissell and Sacks (2002) have shown that a wide variation in dust levels between samplers located within a few feet (less than about 1.5m) of each other, i.e., fixed sampler was within 18 inches (45cm) to 30 inches (75cm) from the machine operator.
- Similar observations were made when the engineering samplers and real-time monitors were positioned between the front two poles of the CM operator cabin as shown in Figure 2.
- Table 3 shows the results of the four published US coal mine studies and the South African study reported in this paper. Table 3 provides enough information to calculate a mean concentration ratio (F/O) between the fixed location (F) and the CM operator (O).

Published Study	No. of Mines	Mean ratio of Fixed/Personal sample	Relative standard deviation (RSD)
Kost and Saltsman, 1977	6	3.53	0.81
Divers <i>et al.,</i> 1982	1	30.7	0.21
Kissell and Jankowski, 1993	5	4.15	0.45
Listak <i>et al.,</i> 1999	5	3.07	0.59
This Study*	8	7.19	2.41

Table 3: Summary of mean ratio of fixed and personal sample(Source: Kissell and Sacks, 2010)

* This is discussed in the next section of the paper.

Based on the above Table 2, it can be concluded that the studies fail to meet the 25% accuracy criterion for which the average relative standard deviation (RSD) is 0.89 reinforcing the view that the fixed-location area samples cannot predict the personal dust exposure of a CM operator.

Derivation of 5 mg/m³ engineering sample limit and its flaws

As noted before, the B7 directive of the DME stipulated an engineering sample compliance dust limit of 5mg/m³ as part of the B7 Directive sampling programme. It is most unfortunate that there was no guidance on behalf of the DMR to provide clarity on the purpose of the sample or on the basis of the 5 mg/m³ limit. There has been confusion between this limit and the personal exposure limit value of 2 mg/m³. Despite this, there were ad-hoc suggestions that the engineering sample limit value was based on the regulated personal OEL of 2mg/m³ for coal dust. Based on the personal communications at the time (Rowe, 1997), it was noted that the 5 mg/m³ engineering limit approximates the personal exposure limit of 2 mg/m³ due to an estimation that the CM generally operates for only 40% of the shift, i.e., 192 minutes. A further assumption was made that there would

be no further exposures to coal dust following the completion of the cutting cycle of the CM despite knowing that the fresh intake air dust levels in the travel road also contain respirable dust. It is also to be noted that the derivation of the value or its assumptions as applied in South Africa is not practiced in other parts of the world.

Engineeing Dust level =
$$\frac{(\text{Personal dust OEL + Intake dust level})*480}{40\% \text{ of CM production period} = (0.4*480)}$$
(1)

In the above equation, the DMR (Rowe, 1997) had assumed that the CM cutting time in a shift is 40 % of 480 minutes, i.e., 192 minutes, that the intake dust level to the CM section would be 0.0 mg/m³ and that the personal occupational exposure limit for coal dust is 2 mg/m³. Substituting these values in the above equation would result in an engineering sample limit value of 5 mg/m³. What can be ascertained is that there is no other scientific rationale for the value of 5 mg/m³. For example, if the production period is increased to 50% or 240 minutes, the expected engineering dust levels should be reduced to 4 mg/m³ without changes to the engineering control system. Based on this background, it is to be assumed that under ideal conditions, the ratio of engineering and personal sample is 2.5, i.e., 5 mg/m³ divided by 2 mg/m³. There seems to be no other scientific reasoning beyond the mere assumption of the CM cutting cycle duration and fresh air dust concentration levels of zero mg/m³. Figure 8 shows the relationship between the engineering CM sample limits for various production periods. With the suggested reduction of coal dust OEL to 1.5 mg/m³, application of the above concept would result in an engineering CM sample limit of 3.75 mg/m³ for a 40% CM cutting time in a shift.





Based on the reported results of engineering sample values well below 3 mg/m³, would indicate that only one of the following is true, i.e., the production period is increased or the sampling location is different. What is therefore certain is that the engineering sampling location in this context is simply not representative of the effect on the remote operator. Therefore, considering that the sample location, as identified, is the same, achieving an engineering concentration of 2.5 mg/m³ would entail the assumption of a production time of nearly 80% of the shift (6.4 hours or 384 minutes) which, from current experience, is not justifiable. In addition, the basis of the initial calculation used to derive the 5 mg/m³ limit, assumed that the shift lengths were in the region of 8 hours and cutting times around 40% of the shift. However, the derived engineering sampling results are extrapolated to full shift duration and sometimes up to 540 minutes. The above evidence suggests that the engineering sample values as stated in Directive B7 cannot be compared to the personal respirable coal dust OEL of 2 mg/m³.

Establishing an accuracy criterion and statistical analyses

The need for an exhaustive pair-wise statistical comparison was not carried out as the available data show glaringly obvious conclusions that the engineering to personal ratio not equal to 2.5. As part of this evaluation, an effort was made to validate if the measurement made using 'engineering sampling' technique will provide what the remote CM operator is exposed to breathing, i.e., personal exposure. Due to the presence of dust gradients represented by the ratio between engineering and personal samples if the operator was located in the cabin, one could have attempted to establish a relationship for an engineering sample's possible use as a proxy for the personal exposure value. In order to establish the engineering sample accuracy criterion standards, for all comparison purposes, the dust level measured by the 'personal CM operator sample' was considered the "true" exposure level. Therefore, the concentration ratio of the "engineering sample" to the 'personal CM operator' sample was calculated. If the variability in the concentration ratio is small, then one can consider accepting the "engineering sampler" for its continued use.

In order to validate the use of an engineering sample as a value for personal exposure assessment, the NIOSH accuracy criterion of $\pm 25\%$ (Kennedy *et al.*, 1995) and the European Community standard accuracy criterion of $\pm 50\%$ (CEN. 1994) is used. As part of the data analyses, concentration ratios of engineering and personal dust level of pairwise data from 8 different mines were created with the denominator being the personal exposure value and the numerator being the engineering sample value. The average and standard deviations of the concentration ratios of each of the mines was calculated. The corresponding Relative Standard Deviation (RSD) values were obtained and compared with the accuracy criterion to validate the acceptance or rejection of NIOSH/CEN accuracy standards. The RSD was calculated from the standard deviation and the mean concentration ratio. In a normally distributed data set 95.45% of the measurements fall within \pm two standard deviation (or \pm 1.96*standard deviation). This would represent that for the \pm 25% criteria with a mean value of 100, then 1.96s = 25 and s=12.755. Therefore, the RSD value for the \pm 25% criteria is equal to 0.127 or less. Similarly, for \pm 50% criteria with a mean value of 100, then 1.96s = 50 and s=25.51 and the RSD value for \pm 50% criteria is equal to 0.25 or less.

Table 4 show the summary statistic of dust concentration levels for the 200 pairwise engineering-CM operator data obtained from 8 different mines over two different periods spanning two decades as an assessment of the benefits and shortcomings of using engineering sampling. Ideally, as per the DMR directive (1997), the ratio of engineering sample to CM operator sample at the current coal dust OEL is 2.5. From the summary statistics (Table 4), it can be seen that there is an absence of any relationship between the two sampling techniques and any useful value in using engineering samples for this purpose. For example, for Mine A, the average of ES/PS ratio is 6.99 against the ideal value of 2.5. Overall, the CV of the ratio between the sampler dust concentrations was above the NIOSH and CEN accuracy criteria.

Mine	Α	В	С	D	Е	F	G	Н	Overall
PS mg/m ³	2.14	1.78	0.72	2.90	1.66	2.58	2.03	0.79	2.00
ES mg/m ³	2.43	2.86	3.45	3.44	3.97	3.05	2.77	4.05	3.00
# of samples	63	50	6	29	21	8	13	10	200
Avg. ES/PS Ratio	6.99	10.72	5.25	7.30	4.88	1.80	2.14	7.41	7.19
Min. ES/PS Ratio	0.06	0.02	0.24	0.02	0.23	0.30	0.60	0.24	0.02
Max. ES/PS Ratio	118.00	86.00	12.73	108.22	31.38	5.03	10.19	36.24	118
SD	20.91	18.45	4.82	20.88	7.50	1.52	2.57	12.05	17.36
RSD	2.99	1.72	0.92	2.85	1.54	0.85	1.20	1.63	2.41

 Table 4: Accuracy criteria for the mean concentration ratio between engineering and personal samples

* PS-Personal CM sample; ES-Engineering CM sample.

COST OF SAMPLING AND USE OF NIOSH PDM3700 REAL-TIME MONITOR AS AN ALTERNATIVE TO CM ENGINEERING SAMPLE

Dust sampling and exposure assessment is part of an expensive pathway to eliminating dust related lung-diseases. As required by regulations and directives, if properly carried out, exposure assessment can result in significant benefits to the mining industry. Uncertainties in mandatory sampling requirements and failure to review vast sample data sets has led to mines continuing to collect many thousands of samples (personal or engineering) with significant annual costs. A paper by Belle and Thomson (2005) estimated the cost of personal and engineering dust sampling for the coal mining industry. In addition, a high level costing of dust control and ventilation was also carried out using models that excluded the cost of the human resources utilised in the sampling programme. From the sampling-control cost model it was evident that the cost of monitoring in the coal mining industry is higher than the dust control costs. Therefore, it is evident that focusing the financial resources on dust control rather than monitoring would surely benefit the industry further by reducing exposure. In addition, the use of new technologies may still assist in meeting the sampling requirements.

In summary, in the absence of a meaningful relationship between the personal dust exposure and engineering sampling and wastage of resources, this study suggests that continuation of the CM sampling has no relevance. An alternative system to the engineering dust sampling program can be using permanent 'real-time' monitors at appropriate locations such as section returns and to take remedial actions using established models on dust levels in coal mine sections. In this regard, the studies on evaluating the use of real-time monitoring such as PDR1000, Hund, SKC Split-2 was carried out by the MHSC (Belle, 2002) in multi-commodity mines. The PDR1000 is the evolution of MiniRam (based on the light scattering principle) commonly used by the USBM/NIOSH mine studies in the 1980s and used as an engineering tool with a correction factor. The PDR1000 has been used as a Passive Sampler was evaluated in South African Coal, Gold, Platinum, Diamond mines to understand the engineering controls and not as a compliance tool. It should also be considered that the PDM3700 is an active real-time sampler that operates using a Higgins-Dewell (HD) size classifier prior to the dust concentration being read by the TEOM. This results in it being one of the best realtime dust monitors that can provide both the effectiveness of the engineering controls as well as personal exposure levels. Other benefits of the PDM-3700 are that it provides real-time temperature data, i.e., DBT and WBT and it is used as a real-time DPM sampler with NIOSH DPM Filter (Gillies, Wei and Belle, 2014).

A total of 955 samples were collected by a coal operator in the concurrent use of both a personnel worn PDM3700 and a gravimetric sampler. This data set supported the NIOSH results. The data shows that the average concentration measured by the gravimetric method (0.83 mg/m³) was virtually identical to the PDM3700 with an average value of 0.82 mg/m³. Scientific studies conducted by NIOSH demonstrated the suitability of the PDM3700 to perform as a compliance personal sampling monitor or if required as an engineering sampler in CM section returns. Therefore, the choice of the PDM3700 personal real-time dust monitor for personal dust exposure management and as a replacement to engineering sampling is definitely a leading practice in the management of the dust hazard. It is recommended that the appropriate SABS certificates for its use in South African operations are obtained and that this device and a new sampling strategy are introduced into our mines.

CONCLUSION

The introduction and intervention by the DME (1997) directive has led to significant positive changes to dust control systems employed in South African coal mines. Previous analysis (Belle and Phillips, 2003) of the CM engineering dust levels indicated that the mere application of the 12 m rule on its own does not solve dust problems, but meticulous and regular maintenance and application of available state-of-the-art dust control technologies, effective dust control strategies and best practices, will ensure the reduced worker exposure. It also identified that the introduction of remotely

controlled CMs in the section would effectively lower the duration, severity and intensity of workers' exposure to coal dust.

Currently, South Africa is the only mining country where regulator prescribed engineering samples are collected every day and in some places sampling even exceeds the requirement. The basis of the engineering sample limit value of 5 mg/m³ does not have any scientific basis for relating it to the personal exposure levels. While the original rudimentary based engineering sample directive has assisted the industry to seek innovative dust control solutions, they do not provide any value in exposure assessment. Therefore, they must be reviewed with the objective of its discontinuation, as there is no relationship between engineering sampling and personal sampling the motive behind its continued use has to be seriously questioned, especially where resources are scarce.

Based on this investigation, it is considered that the continued use of CM engineering sample results is promoting misinformation about personal dust exposure of CM operators and multiplying the confusion on engineering sampling techniques. As part of this study, an attempt was made to identify the relationship between daily CM engineering sample and the CM operator personal exposure values. This paper has clarified the myths about sampling frequency benefit ratios and identified shortcomings and improvement opportunities in the management of the exposure levels in underground coal mines and these are summarized below:

- The study has demonstrated that the fixed-location CM engineering sample cannot predict the personal shift dust exposure of a CM operator. The South African study has shown that the average RSD value of over 2.41 against a measured value of 0.58 from the US coal mine studies demonstrates that engineering samples are totally unsuitable for any personal exposure assessment. Based on two decades of sampling and its poor correlation between engineering and personal samples, it is recommended that engineering sampling should be discontinued and replaced with measurement of remote operator personal exposure data that provides superior information towards dust exposure management.
- The commonly held belief that the engineering sample results arean indication of coal dust explosion risk is totally flawed.
- The performance of ventilation and dust control systems in a section can be achieved by current operating alternatives such as on-board methane sensors, CM water and pressure flow monitoring devices, scrubber monitors, section intake and return real-time air velocity monitors in addition to regulatory manual check-lists, start-up shift inspections and standard operating procedures.
- From an analyses of the cost of monitoring and engineering controls, it is evident that the
 operating cost of dust monitoring in coal mining industry is higher than the dust control costs.
 Therefore, it is essential that efforts be made to re-focus financial resources into dust control as
 this would result in reducing exposure and the use of real-time monitoring would equally meet the
 sampling requirements.
- The currently held perception that the 1997 DMR directive with an inherent requirement to
 operate the CMs with on-board scrubbers is flawed and, in addition, creates a continual noise
 hazard at the coal face. This particular requirement has prevented the coal mining industry from
 considering other leading practices, such as exhaust ventilation or force-exhaust ventilation
 systems as practiced elsewhere in the coal mining world. For example, coal mines in both the
 USA and Australia continue to operate up to 125 m long development headings using exhaust
 ventilation systems in managing very gassy and pre-drained, dusty coal seams.
- It is suggested that with the availability of new real-time technology such as the NIOSH PDM3700 engineering sampling can be replaced with personal exposure monitoring and effectively utilize the value add personal exposure assessment results to also evaluate the engineering controls.
- The NIOSH PDM3700 is a well proven personal –real-time dust compliance monitoring tool that is currently used by MSHA for compliance monitoring and has been introduced in Australian coal

mines. The PDM3700 not only provides an indication of the engineering control performance but also provides a valid sample for quarterly regulatory dust data submissions for other Homogenous Exposure Groups (HEGs) and critical occupations such as the roof bolt operator and shuttle car operator.

In summary, the focus of this paper is to review two decades of CM engineering sampling experience, perceptions and improvement opportunities. In addition, the status of daily collection of engineering samples, their subsequent use and relevance in managing coal dust exposure and explosion risk management is shown to be questionable. The paper highlights the strategic opportunities that could be better employed to improve the workplace in South African coal mines. It is suggested that the knowledge demonstrated in this paper be used to drive step-changes in sampling requirements by introducing flexibility for mines in approaching the measurement and management of personal exposure in coal mines.

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The data analyses carried out for this paper was part of an independent investigation by the author who was part of development of CM engineering dust controls and the introduction of ISO/CEN/ACGIH curve in South Africa. This work is dedicated to South African coal mine workers, regulators and those in the industry who are striving to achieve improvements in the occupational environment. The author is also indebted to Prof. H. Phillips of Wits University and other critics in improving the quality of the paper.

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THE CASE FOR DEVELOPING AN AUSTRALIAN TECHNICAL SPECIFICATION FOR STRUCTURAL DESIGN OF VENTILATION CONTROL DEVICES

Michael Salu¹ and Verne Mutton²

ABSTRACT: Ventilation management and control plays an essential part in underground coal mining. Failure of the ventilation system or failure of underground seals can lead to multiple fatalities and closure of a mine. Examples include: Moura No.2 (Qld, 1994); Sago (USA, 2006) and Pike River (NZ, 2010).

Following the Moura No.2 disaster, the Qld Dept. of Mines put new regulations in place specifying pressure ratings for various classes of Ventilation Control Devices (VCDs). They also initially specified that only VCD's that had been subject to "full scale testing" would be accepted for use in Qld mines. No guidance was provided on how the full scale test results were to be applied to the design of VCDs in the field. It is considered that an Australian Standard for VCDs should be developed to address commonly observed issues including:

- Factors of safety
- Design methodologies and designer qualifications
- Material properties, testing and verification
- Dual ratings for overpressure and water head
- Provision for inclusions such as access hatches, doors and pipes.

Australian Standards are extensively researched, peer reviewed and subject to public comment. The entire process typically can take from two to four years. An Australian Technical Specification is a one-tier lower document than an Australian Standard, produced by an expert committee on the basis of consensus. Although peer reviewed, it is not subject to public comment and could be completed within 12 months.

It is suggested in light of the critical importance of VCDs and the lack of any Governmental or Regulatory technical progress since 2001, that the Coal Mining Industry should pro-actively assemble an expert committee and prepare a business case to Standards Australia for development of an Australian Technical Specification for VCDs with a target completion date of June 2018.

INTRODUCTION

The mining of coal underground has historically (Department of Labour) been recognised as one of the more hazardous occupations in the world. It is a universally recognized principle of underground coal mine safety that there must be proper ventilation of the mine. Indeed, no aspect of safety in underground coal mining is more fundamental than proper ventilation.

Before developing an argument for supporting the development of a design standard for Ventilation Control Devices (VCDs) it is pertinent to suggest a definition for seals and stoppings. It could be argued that the following 100 year old definition for a seal is still relevant today and literature provides many similar definitions.

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Old Definition (Herbert Bucksch, 1998): "a tight seal partition or barrier of wood, rock, mud or concrete in mines for the protection against gas, fire and water."

Seals are used extensively in mining to isolate worked-out areas and active fire zones. Stoppings separate the air streams in intake and return airways. It is worthwhile examining how VCD designs have evolved by examining in greater detail the milestones globally that have driven design changes in the last one hundred years. Explosions (Mutton and Remennikov, 2010) of gases and of coal dust have always been a basic hazard in coal mines and to this day continue to be the cause of disasters in coal mines where ventilation control is inadequate. The investigations and advancement of knowledge in VCD design and construction has tended to be driven by these disasters. There has been no significant review of VCD design in Australia since the Task Group 5 investigations after the 1994 Moura disaster. Following the Moura No.2 disaster, the Qld Dept. of Mines put new regulations in place specifying pressure ratings for various classes of VCDs. This prescriptive legislation requiring the construction of designed VCDs provides some level of insurance against an explosion breaching seals and stoppings. The damage incurred on a VCD by an explosion will be influenced by the magnitude and shape of the pressure-time curve. This is recognised by the coal industry in the United States following the enactment of the 2006 Miner Act and MSHA's issuance of the Emergency Temporary Standard (ETS) after the 2006 Sago disaster. There is now a set of guidelines, the "final rule" issued in 2008 for mine seal design (MSHA, 1996). During this period extensive research investigating various mine geometries and a range of fuel loadings showed that an increase in seal design load capacity was necessary to adequately protect mine workings against explosions.

Today the designer has much greater access to a variety of construction materials, live testing and numerical design methods. Mine operations provide the designer with the required overpressure ratings for any VCD based on legislation. Unless the designer has an understanding of the duty requirements of the VCDs and of the construction materials and methods then ventilation controls may become ineffective during their life-cycle. However there are still VCD design challenges due to many variables such as a requirement for dual design loads, barometric changes, transient loads and the changing condition of the incumbent strata. Despite the ability to live test VCDs and provide numerical design modelling there will always be a statutory requirement to systematically inspect VCDs in the mine operation and make provision for repair. There are many design challenges including: at what condition does the VCD no longer comply with its design rating and what repairs are necessary to reinstate the rating.

It is left to the individual mine operations to determine whether these over pressure ratings are sufficient to protect the mine workings. Gas monitoring and inertisation practises in Australian longwall coal mines minimise the possibility of the goaf passing into the explosive range. This paper will not investigate the level of protection that is required. An Australian technical specification for structural design of ventilation control devices would provide a design guideline regardless of the duty required of the VCD.

The Qld Dept. of Mines (Mutton and Salu, 2013) initially after the Moura disaster in 1994 also specified that only VCD's that had been subject to "full scale testing" would be accepted for use in Qld mines. No guidance⁴ was provided on how the full scale test results were to be applied to the design of VCDs in the field. It is now accepted practise by regulators and some structural engineers that VCDs can be designed and rated using numerical methods. Development of numerical models for VCD design from the results of explosion testing can be quite different to design using structural analysis that uses inputs from seal and stopping material characterisation testing. A variety of structural design questions that have arisen during VCD design and certification would be addressed by a working group developing a technical design standard. Dual load ratings, factors of safety and numerical modelling will be expanded upon as a portion of the relevant questions to be addressed. Due to longwall mining retreating to the rise and impoundment of water, many goaf seals must be simultaneously rated for water loads up to 30 metres of water and explosion ratings up to 345 kPa (50

psi). The distinct differences in duty between water bulkheads and explosion rated seals will be discussed with the requirements for a dual rating.

For (Muttonb and Salu, 2013) the purposes of structural design, the loads are considered to be "ultimate" loads using terminology from Australian Standards and the design thicknesses derived from live test results are said to have a "safety factor" *greater* than 1. There are many design variables that will influence the safety factor adopted for VCD designs, with some of these listed below:

- 1. The condition of the roadway in contact with the VCD
- 2. The changing loads due to adjacent extraction that are difficult to define numerically
- 3. Whether the load is transient (explosion) or long-term e.g. water impoundment.

It will be argued that by using the framework provided by Australian Standards a universal standard for the provision of VCD design can be developed for Australian underground mines, overcoming a situation now where consistency of design guidance is uncertain. This would also assist operators to provide clearer definition of designs in commercial tenders. There are many types of standards and it must be decided which is more suitable for providing design guidance. Whatever is developed will require industry acceptance and needs to be completed in a suitable timeframe with the available resources, a challenge where people resources have been depleted in an industry wide downturn.

The challenge provided within this paper is to persuade the mining industry that a VCD design standard is required. Once this is accepted, funding and a motivated team will be required to make it a reality.

HISTORICAL BACKGROUND: THE LAST 100 YEARS OF VCD DESIGN

This discourse is a brief summary of events shaping VCD design that have led to the technology we use today to safely and economically control ventilation flow in underground mines. The Laurium silver mines of Greece, operating in 600 BC (Mutton and Remennikov, 2010), had layouts which reveal that the Greek miners were conscious of the need for a connected ventilating circuit. At least two airways served each major section of the mine and there is evidence that divided shafts were used to provide separate air intake and return connections to the surface. The first great textbook on mining, De Re Metallica, was published in 1556 in Latin by Georgius Agricola, a physician in a thriving iron ore mining and smelting community of Bohemia in Central Europe. A number of the prints show ventilating methods and controls that include diverting surface winds into the mouths of shafts, wooden centrifugal fans powered by men and horses, bellows for auxiliary ventilation and air doors.

Much historical literature on coal mining concentrates on the effects of the poisonous and explosives gases found in coal mines. From the seventeenth century onwards, papers began to be presented to the Royal Society of the United Kingdom on the explosive and poisonous nature of mine atmospheres. The onset of the industrial revolution and a rapid rise in the demand for coal had precipitated many disastrous explosions from methane and coal dust due to lack of understanding of the necessary ventilation controls and the nature of the combustible materials. In metalliferous mines many miners succumbed to the effects of "gas" i.e. carbon dioxide generated from oxidation of minerals and accumulated spoil in the workings. John Buddle (1773-1843), a mining engineer in northern England developed improved methods of ventilation control including providing layouts for discrete panels and separate intake and returns i.e. the first parallel circuits (McPherson, 1993). Decades later Atkinson in 1854 proved mathematically the improved airflow and reduced concentrations of methane from these circuits and modern ventilation theory was born.

In response (Mutton and Remennikov, 2010) to the alarming number of fatal explosions and fires in U.S underground coal mines the Bureau of Mines was set up on July 1st, 1910 (Tuchman and Brinkley, 1990). Likewise in Poland, from 1925 Experimental Mine Barbara conducted live tests on mine seal designs typically constructed in coal mines. Various other experimental mine facilities

around the world e.g. Buxton (Great Britain) and Tremonia (Germany) also conducted live explosion tests in the absence of mathematical models that could adequately describe seal response to such explosions. Mine disasters in 1933 and 1960 prompted UK Commissions which reported it desirable that explosion proof stoppings be designed to withstand explosion pressures of 20 to 50 psig (140-345 kPa).

In 1980s Europe monolithic seal designs were being developed using pumpable (Underground coal mining safety research materials) such as concrete, Gypsum, anhydrite, Flyash and Bentonite replacing the rocks, bricks, timbers, sand, dust, and cement which had been commonly used for seals. During this time, gypsum based products were preferred by British, Czechoslovakian, and German coal miners who believed them to be the most effective, easiest to use, and least costly. In1981 the National Coal Board developed a two component high yield grout that was used for gate-side packs on advancing longwalls and for seals.

It was in 1930 that experimental work involving measurement of seal response to explosions was carried out by the U.S Bureau of Mines. This was the beginning of understanding structurally what influenced the performance of ventilation seals when subjected to an explosion overpressure. There was also no means to physically measure and define seal response to real time explosion impulses. In the USA sealing unused and abandoned areas was a common practice in coal mines prior to World War II. The few seals (bulkheads) since built were principally in areas having a potential for spontaneous combustion. Historically, "explosion proof" has been a consensus interpretation. For example, the 1921 regulation for sealing connections between coal mines on U.S. Governmentowned lands required that stoppings withstand a pressure of 345 kPa (50 psig). This was the earliest known standard for seals in the United States and the main concern during this period was that sealed areas needed protection from ignition of gases that emanated from working areas. In the USA the Federal Coal Mine Health and Safety Act of 1969 required that such areas be ventilated or sealed with explosion-proof bulkheads. Later research at the United States Bureau of Mines (USBM) showed that seldom, however, do pressures 200 feet and more from the origin of an explosion exceed 140 kPa (20 psig) unless coal dust accumulations are excessive and the incombustible content of the dust is less than required by law.

In the United States, since 1971, statute 39 CFR 75.335 (Mine Safety and Health Administration -Title 30 Code of Federal Regulations, 1997) had required a seal to "withstand a static horizontal overpressure of 20 psi (140 kPa). Since 1986 there had been 12 known explosions within the sealed areas of active United States coal operations where seals were destroyed. In some of these explosions lightning was suspected as the ignition source. Later events in 2006 would put the 1971 statute into question. It was also recognised by 1990 explosive mixtures could form behind the seal because of leaking air from the working areas. There was potential for explosion pressures greater than 20 psi (140 kPa) with ignition sources from falls of roof or spontaneous combustion in the goaf. The Mine Improvement and New Emergency Response Act ("MINER Act") required the Mine Safety and Health Administration (MSHA) to increase this design standard by the end of 2007 due to tragic accidents at Sago, WV and Darby, KY Mines in 2006 caused by methane explosions behind sealed off areas (Mutton and Salu, 2013). Following the enactment of the 2006 Miner Act and MSHA's issuance of the Emergency Temporary Standard (ETS) more stringent performance standards have been adopted for mine ventilation seals (Mutton and Remennikov, 2010). There is now a minimum standard of 345 kPa (50 psi) (designed, constructed and maintained) for a specific pressure-time curve, when the atmosphere inside the sealed volume is monitored and maintained inert. In the United States more commonly pressure rated seals have a capacity for 827 kPa (120 psi) in line with the findings of the NIOSH study entitled, "Explosion Pressure Design Criteria for New Seals in U.S. Coal Mines" (Zipf, Sapco and Brune, 2007). The findings of this report have challenged globally established beliefs in seal design and explosion propagation. The research that followed these disasters provides guidance on the scenarios that will generate explosion pressure/time impulses.

Extensive use of computer aided numerical analysis provided a range of designs for use in the USA using a variety of materials over a wide range of explosion pressures.

Many seals and stoppings, although designed as explosion rated, often unintentionally impound heads of water. Seals expected to withstand a hydraulic head (bulkheads) have unique design characteristics. The loads are often long term (not transient) and the condition of the strata holding the seal is critical and deterioration and leakage can occur over time. Much emphasis is put on designing strata injection programs to seal the surrounding strata. Care must be taken that when adjacent mining increases vertical abutment load, these loads do not severely impact the bulkheads stability. Designs must have the ability to impound water where subsidence breaches surface aquifers. With longwalls extracting to the rise the use of water holding bulkheads is increasing.

After the Moura No 2 Mine explosion in 1994 (Mutton and Remennikov, 2010) there were changes to mining legislation in Queensland that required all VCDs to have been tested in an internationally recognised laboratory. In 1997 at Lake Lynn Experimental Mine, Tecrete Industries (designed an explosion seal and stopping test program in which instrumentation was introduced for the first time for measuring the structural response of VCDs when subject to transient loads (Weiss, Cashdollar, Mutton, Kohli and Slivensky 1999). Using linear variable transducers, accelerometers and carefully situated overpressure monitoring, live test data was captured that would be useful to enable the design of VCDs in a wide range of roadway sizes and pressures ratings ranging from 14 kPa (2 psi) to 345 kPa (50 psi). Using this data, numerical models were developed to facilitate the design of a wide range of VCDs. There have been various live test programs since 1997, which provided measurements of structural response that could be used to develop engineering design tools for a range of VCD designs. Design is now undertaken with sophisticated computer based numerical analysis. However building representative numerical models (meshes) with suitable material properties and interpreting the results of analysis requires considerable experience and structural knowledge. Although most VCDs in Australia are now designed by professional engineers, there is a variation in design practises and outcomes with no design methodology common to the coal industry in Australia.

STANDARDISATION OF VCD DESIGNS AND WHY IT IS IMPORTANT

Standards generally go unnoticed (Gibbons, et al, 1992)

"They are mostly quiet, unseen forces, such as specifications, regulations, and protocols that ensure that things work properly, interactively, and responsibly".

Development of standards came about during global industrialisation at the end of the 19th century. Now standards govern the design, operation, manufacture, and use of nearly everything for products that are produced around the world. Construction of VCDs in the underground coal industry and metalliferous mines is typically either carried out by contractors or in-house workers. Design certification in Queensland and New South Wales coal mines is provided by Registered Professional Engineers Queensland (RPEQ). In-house development of VCD construction techniques and operating procedures results in a variety of outcomes for mining companies. As there is no agreed or available standard there are currently widely different design methodologies and design approaches used.

What is a standard?

The International organization (ISO/IEC GUIDE 2:2004) for Standardisation (ISO) and Electrotechnical Commission (IEC) Standard (ISO/IEC) Guide 2 defines a Standard as a:

"document, established by consensus and approved by a recognised body, that provides, for common and repeated use, rules, guidelines or characteristics for activities or their results, aimed at the achievement of the optimum degree of order in a given context. Standards should be based on the consolidated results of science, technology and experience, and aimed at the promotion of optimum community benefits."

Standardisation (according to Xie, *et al*, 2016) is the process of implementing and developing technical standards based on the consensus of different parties that include commercial organisations users, interest groups, standards organisations and governments. In a more general sense, standards help to codify best practices, methods and technical requirements, contributing to the community demand for a safe and sustainable environment, a point not lost on the coal mining community. Standards are intended to promote compatibility, interoperability, quality and safety. It is very important that VCDs consistently perform the way they were intended to over their life span.

For engineers and ventilation practitioners a standard would define quality and safety criteria in a common language setting achievable goals in a practical way. A Standard or Technical Specification would provide a platform for regular reviews which would be required to keep pace with technological advances. Use of design standards is common place in the building and construction industry with government, industry and the community as stakeholders. Of the VCD design research including live testing instigated after mine disasters the question must be asked; is this material reviewed and updated at regular intervals? The outcome from developing a "consensus" standard or design is that any design innovation has a platform for inclusion enabling the ideas to be used by the whole mining industry. The development of a standard is a rigorous, structured and formal process that can satisfy the requirements for:

- 1. Voluntary or mandatory applications (achieve as a minimum objectives of safety, quality and performance)
- 2. Regulatory compliance
- 3. Contracts
- 4. Guidance

Standards may provide guidance for: products' definition, materials, analysis and design, material testing, construction and maintenance. As an entity a standard has no legal status or requirement for compliance. It can be cited in legislation and commercial contracts. A recent example is the NSW code of practice: Strata control in underground coal mines, which is an approved code of practice under section 274 of the Work, Health and Safety Act 2011(NSW Government 2015).

STRUCTURAL DESIGN QUESTIONS FOR THE CODE COMMITTEE OR ITS WORKING GROUP

It is suggested that the following topics could provide a suitable starting point for issues to be addressed by a technical committee or working group:

a. Scope

The suggested scope for a Technical Specification (TS) is to cover structural design of VCDs, including VCDs that may also impound water (bulkheads). The VCDs will be limited to those constructed in underground drifts and roadways i.e. anchored all around by roof, ribs and floor strata. It is not intended to include underground dams, although they could be added later. Aspects such as inspection, construction and maintenance would only be addressed to the extent that assumptions required for design purposes, need to be specified.

b. Materials of construction

Traditionally, a wide variety of materials has been used for VCD construction underground. The purpose of a Technical Specification is not to limit materials or techniques but to provide scientific guidelines (known as performance specifications) that define the expected performance characteristics. For example, a VCD should be resistant to fire as well as to overpressures. Some

materials are better suited for long-term durability or to withstand strata convergence and a TS will provide guidance and "best practice" as well as minimum structural requirements.

c. Types of underground structures

Initially, the TS should be limited to the more common and arguably most straightforward types of underground VCDs, including: stoppings, seals, overcasts and air locks. Dual-function VCDs, designed to impound water as well as resist blast pressures are also reasonably common and have many similarities with VCDs. Other, more exotic structures such as so-called "coffin seals", machine doors, water storage dams and high-head bulkheads are seen as being outside the scope of the initial TS.

d. Use of material testing data and live testing data in numerical modelling

Full-scale live explosion testing performed both in Australia and overseas has demonstrated that the structural behaviour of VCDs subjected to a short burst of high pressure while being fully confined by the surrounding strata is complex and does not follow any simple structural rules. A combination of complex structural behaviour and varying material properties when subjected to shock loading means that it is difficult to extrapolate the usual small scale test results (for example from slowly crushing concrete test cylinders) to real VCD behaviour.

A wide variety of approaches that have been applied to this issue over the years results in an equally wide variety of outcomes. Modern computer simulations are able to use limited live test data to produce useful predictive models. However, the approach is generally too time consuming and expensive for routine VCD design tasks. This is a difficult question to resolve, as it is also tied in with commercial interests of VCD suppliers who are understandably reluctant to provide access to privately funded research into this question.

e. Design loads

Currently in Australia where VCD design pressures are specified by State Mining Authorities, they are called up as uniform (and presumably static) pressures. Design pressures called up in Regulations are tabulated below:

Table 1: Statutory design pressures in native units

New South Wales (NSW)	20, 120psi		
Queensland (QLD)	14, 35, 70, 140, 345 kPa		

If the above design pressures are standardised to metric units, the following matrix provides the above information in another form:

State	Statutory Design Pressure (kPa)						
	14	35	70	140	345	830	
NSW	No	No	No	Yes	No	Yes	
Qld	Yes	Yes	Yes	Yes	Yes	No	

Note that neither dynamic factor nor pressure/time history is stipulated by either authority. Current industry practice appears to be to adopt the above specified pressures as nominal uniform static pressures. That is, they are not considered "explosion pressures" for design purposes. Recent research in the USA has shown that dynamic and confining effects can significantly amplify

explosion pressures. Pressure/time curves have been produced for both gas and coal dust explosions, based on that research. The TS committee should consider how design loads are to be interpreted and then codify their findings to eliminate uncertainty. This process may, in turn, lead mining regulators to review VCD design overpressures in light of recent overseas research recommendations.

f. Factors of safety

Questions regarding Factors of Safety (FoS) to be applied when designing VCDs are one of the most common issues raised by Ventilation Officers and others responsible for VCD construction. This would be one of the most important issues to be addressed by a TS and a single clear set of guidelines would provide significant benefits to the underground mining industry. As an example, in late 2016 a major Australian coal mine released a tender for VCD and bulkhead design and construction covering a wide variety of roadway sizes and four (4) pressure ratings. Thicknesses (and costs) were requested for each combination, a total of some 30 different VCDs. However, there was no mention anywhere in the 20-page tender document of what factor of safety (if any) was required for any of the VCDs or bulkheads. In response to a query regarding the FoS for this tender, the mine initially didn't know and then appeared to hedge their bets by specifying a range of different FoS to be applied to all of the VCDs and bulkheads. This resulted in a tripling of the amount of work required to calculate all of the requested data, for each of the prospective tenderers. The selection of agreed FoS will be a challenging task for a TS committee but there is a wide range of literature available for guidance, including existing Australian Standard AS1170. Decisions on an appropriate FoS must be made based on a sound engineering basis, so that they can provide a balance between cost of implementation and risk of failure.

g. Design life and importance

The expected design life for a VCD will influence its cost, as a short-term structure can usually be more lightly constructed than a permanent structure with the same level of overall safety. There is no known information on any current design life guidelines or requirements for underground structures other than seals which are to be designed "for the life of the mine". In central Queensland, several mines were originally designed for a twenty year design life in the 1970s and 1980s. These mines are currently still operational due to improvements in mining technology over that period. They now face the challenge of infrastructure that is 40 years old as well as the question of what design life should be specified for new structures.

As previously described, there are a variety of VCDs in underground mines, performing various tasks. These can range from directing ventilation to permanently sealing off goafs or adjacent workings. An "Importance Factor" (IF) is one concept that structural engineers have used in other fields to help quantify these types of differences. The IF could include an allowance for the expected design life and would potentially form part of the calculation to determine the appropriate FoS for a VCD or underground structure.

h. Structural behaviour

As discussed briefly in Item "d" above, full-scale explosion testing of VCDs has shown that these devices exhibit complex structural behaviours. Depending on their configuration, VCDs can resist blast pressures using a variety of mechanisms (Pearson, *et al*, 2000):

• **Sail** Flexible so-called "sail" stoppings act purely in tension just like a sailboat sail (Figure 1). These are typically only suitable for low-pressure stopping applications.

- Plate VCDs constructed from thicker material will tend to flex slightly under load and carry blast loads by bending. (Figure 2). Such VCDs require significant flexural strength and/ or thickness to carry loads.
- Flat Arch as VCDs increase in thickness, the bending mechanism starts to give way to an arching mechanism (Figure 3) where the loads are carried primarily in compression rather than bending.
- **Plug** Relatively low compression strength materials can be used for VCD construction if they are thick enough. These form a plug, which resists loads through direct compression and internal shear strength as well as shear resistance along the roadway contact. (Figure 4)



Figure 1: Sail mechanism

Figure 2: Plate mechanism (Pearson)



Figure 3: Arch mechanism (Pearson)



In practice the plate, arching and plug mechanisms form a continuum of varying structural behaviour, which makes simple modelling unreliable. However, through parametric studies, guidelines are available in the literature regarding the transition points between these systems. A TS committee could develop simplified rules using, for example, height to thickness ratio to provide guidance to engineers for structural modelling of VCDs. As mine drifts and roadways are typically limited to between 2.5 to 4.5 m high and 5 to 6 m wide, the work required to perform such a study would be well within the scope of an ACARP-sponsored report.

i. Design and analysis methods

Several Australian Standards (AS) covering structural designs provide for a "tiered" approach to structural analysis and design. In other words, there is allowance for simplified methods to be

used to produce well understood routine designs. For more complex or unusual designs, guidelines are provided in those AS for more detailed methods of analysis.

A TS for structural design of VCDs could similarly provide a simplified design section, perhaps limited to stoppings or to short-life structures. Some design codes provide design charts and tables for routine design tasks and it may be possible to incorporate similar information into a TS for VCDs.

j. Designer qualifications and responsibility

Currently, Queensland Mining Regulations require that VCDs are designed and certified by a Registered Practicing Engineer, Queensland (RPEQ). In NSW, the requirement is for VCD's to be designed "fit for purpose", which is a vague term to use for something as important as a VCD. There is also a requirement in NSW15 for "Structural rating (of seals) to be to a standard acceptable to the Chief Inspector of Coal Mines".

It is suggested that design responsibility for VCDs should rest with a suitably qualified and experienced structural, mechanical or mining engineer. The Institution of Engineers, Australia (I.E. Aust.) already provides suitable oversight of engineering qualifications and experience at a professional level, which provides the starting point for this section. A designer should be able to demonstrate a body of work, relevant to design of VCDs, which would provide a level of assurance to mines that the "experienced" part of the "qualified and experienced" requirement was achieved. It may be that Mining Schools could provide short courses in VCD and bulkhead design to meet potential demand for appropriately trained engineers? Another option could be to recommend that for critical life of mine VCDs with a pressure rating greater than say 300 kPa, an independent engineering design review should be performed prior to construction of the VCD.

k. Geological/Geotechnical issues

Geotechnical aspects of VCD design are very similar to geotechnical issues for footings of surface structures. Provision of geotechnical engineering advice for VCD design is generally provided by mining geologists. In some cases, their exposure to structural engineering has been limited and a TS could include a section outlining the geotechnical information that VCD designers require.

A working underground mine is a dynamic, changing environment and geotechnical factors will change over time even though the overall geology of a mine site is fixed. VCD and bulkhead effectiveness can be very heavily influenced by local, minor geological features and therefore a pre-construction site inspection by an experienced mine geologist is essential prior to construction of long-term or high-importance structures. Some of the issues requiring geotechnical/geological advice include:

- Overall site suitability for the proposed VCD
- Strength of floor, ribs and roof strata
- Likely degree and timing of roof convergence or floor heave
- Potential for gas or water leakage around the VCD (through the strata)
- Recommendations for strata injection, if required

I. Robustness and siting

Robustness is not a new concept in design. In the past it has formed a key part of rules of thumb that were widely used for structural design prior to modern physics-based methods. However, over-reliance on physics has led the Australian Loading Codes Committee to formally re-introduce in 2002 the concept of robustness into Australian/New Zealand Standard AS/NZS 1170.0. The

best way to illustrate robustness is using an example of a plaster stopping for a 6 m wide roadway, 4.5 m high, which was reportedly designed (by a qualified engineer) to be 60 mm thick. Putting aside the question of suitability for ventilation pressures, a plaster stopping of that size and thickness has virtually no capacity to withstand accidental impacts from plant, light vehicles or even from loosely stacked materials falling onto it. Specification of minimum thicknesses for plaster, grout and concrete VCDs would be one method of incorporating robustness requirements into a TS. The influence of siting is very important but is sometimes overlooked even by experienced underground miners. The structural capacity of a VCD relies on it being solidly connected to the roof, ribs and floor. This assumption on the strength of the strata can be compromised by the proximity of adjacent workings. Although each situation is different, rules of thumb that are currently used by various mines to specify minimum distances to roadway intersections or niches could be reviewed, assessed and codified by a TS committee to provide a consistent set of guidelines to the industry.

m. Durability

Long-term durability of VCDs due to underground environmental effects is influenced by a number of factors:

- Underground hazards such as strata movement, spontaneous combustion, corrosive mine water, corrosive gasses and accidental impact
- The material properties of the VCD and its fixings (if applicable)
- The applied hydraulic loads in the case of water bulkheads. An explosion in the goaf although not breaching a seal could affect its durability.

As each mine will have a unique combination of durability hazards, these will need to be assessed on a case by case basis. A TS would provide guidance and recommendations based on the best available industry and research information, for each particular hazard.

n. Provision of inclusions such as doors and pipes

The interface between VCDs and access doors of hatches has been a "grey" area for some time, in terms of design responsibility. Typically, VCD hatches and doors are designed, fabricated and tested independently of the VCDs of which they are intended to be part. The VCD designer typically assumes at first that the VCD will be a uniform structure with no openings. In practice, doors and hatches are required in many VCDs, even if they are later intended to be permanently sealed. It may appear to be obvious that for a VCD to be rated to a particular overpressure, a door or hatch should be rated to the same or higher pressure. But there is a need for an appropriate FoS for doors and hatches and any limitations that should be placed on their use and positioning.

It is not uncommon for mines to request two doors to be placed in a VCD. Knowledge is available of an incident where a mines Inspector found a VCD with three doors installed with a subsequent engineering assessment reporting that the overall VCD strength had been significantly compromised. As a result of the engineering assessment, one of the openings was permanently and rigidly sealed in order for the VCD to achieve the required pressure rating.

Cast-in pipes for water or gas drainage and tube bundles for gas monitoring are also required to penetrate VCDs. These are typically a maximum of 150 – 200 mm diameter and do not have as dramatic an effect on VCD strength as doors or hatches. However, a concentration of pipes could create a weak spot in a VCD, potentially compromising its strength. The Qld Mines Inspectorate produced Mines Safety Bulletin No. 107 in 2011, which stipulated maximum pipe sizes and minimum spacing. This appeared to have the intention of minimising the risk of creating a weak point in a VCD through prescriptive means. That document would provide a useful starting point

for a clause in a TS that could be expanded to include doors, hatches and potentially other built-in items.

o. Dual functionality VCD/bulkheads

When a roadway is constructed on a slope, there is always the potential for seepage water to collect on the uphill side of a VCD. Typically this is of minor concern unless the VCD has not been constructed from water-resistant material. Once a goaf is permanently sealed, there are limited options for inspecting the inbye side of a seal. Such seals often have U-tubes fitted to allow drainage of water from behind the seal without allowing gas transfer. The height of the U-tube above the floor will limit the water head behind the seal. However, sometimes valves are also fitted to the U-tubes to control water flows and in those cases water build up behind a seal can be significant in terms of structural loading on the VCD. The short, sharp pressure on a VCD from an overpressure event is quite different to the sustained pressure from impounded water. Water pressure will also be higher at the bottom of a VCD than at the top, compared with uniform blast pressure. Large bodies of impounded water behind seals will attenuate a shock wave from an explosion, however the momentum imparted to that body of water will still effect the rigid body of the seal. There is limited information available in technical or research literature regarding methods for designing VCDs to retain water. In the authors' experience in Australian practice, individual mines will request a variety of FoS for water-retaining VCDs ranging from 1.5 up to 4. Some mines request a dual rating i.e. an overpressure rating and a separate maximum water head rating. Other mines ask for the overpressure rating to be added to the water head pressure and then the total resultant pressure is required to have a FoS applied to it. This can lead to very high design pressures and consequently very substantial and expensive structures. The question of extending the TS work from dual-purpose VCDs to bulkheads intended purely for impounding water will be one to be decided by the committee, depending on available resources and expertise. The authors believe that in the interests of time, at this point standardisation of bulkhead design might be best left to a future update or perhaps a separate publication.

p. Construction requirements

When a VCD is designed, there are a number of assumptions regarding construction that are inherent in the design process. The purpose of specifying particular construction requirements in a TS is to highlight the critical factors that will affect the strength of a VCD. Some examples of these include:

- Use of depth gauges for sprayed VCDs
- Cylinder strength testing for concrete VCDs
- Manufacturers certificates for rated inclusions
- Geologist and mine undermanager or ventilation office sign-off on VCD location

q. Inspection, testing and maintenance

Once a VCD has been constructed in accordance with its design specifications including any drawings, responsibility for maintenance of the VCD passes to the mine. Without proper maintenance, the designed pressure rating of a VCD cannot be guaranteed indefinitely.

Regular inspections must be performed on VCDs during their life, with the frequency depending on the use, life expectancy and importance of a VCD. Regular monitoring will ensure that any developing issues can be addressed as soon as they occur and will significantly reduce the risk of an unexpected, sudden failure. Visual inspections are expected to be the predominant form of regular VCD inspections, as they are at present (when implemented). Several Non-Destructive Test (NDT) methods have been used successfully on completed VCDs and bulkheads, to verify their construction and structural condition. NDT testing could be specified at longer intervals for less important VCDs, or more frequently for operationally critical VCDs.

r. Modification, strengthening and repairs

Any modification, strengthening or repair of a VCD should be carried out under engineering supervision and ideally with input from the original VCD designer. A TS could provide examples of minor modifications, strengthening or repairs that do not require a full engineering assessment. This could include, for example, repair of minor damage to a sail stopping resulting from accidental impact. For more extensive modifications, such as installation of machine doors or strengthening (up rating) a 5psi (35kPa) stopping to say a 20psi (140kPa) seal, these should always be subject to a full engineering assessment similar to that for a new structure.

DEVELOPMENT OF A DESIGN STANDARD

As previously discussed at the start of this paper, production of an Australian Standard is timeconsuming, in part because of the stringent requirements for public consultation and peer review. A Technical Specification is also peer reviewed but does not require mandatory public comments and therefore can be brought to publication more quickly than a Standard. Using a recent (2015) example, the concrete fastener industry produced a 96-page TS within 12 months. They did have an expert committee already formed together with support from both the I.E. Aust. in Victoria and Swinburne University.

Another, lower-tier option is to develop a Mine Design Guideline (NSW) or Mine Safety Bulletin (Qld). These might be quicker and easier to put together but will not achieve the objective of preparing a single unified document that can eventually be used anywhere in Australia.

Developing a Code of Practice (CoP) is another possibility. A CoP is a practical guide to achieving the standards of health, safety and welfare required under various WHS and Mining Acts and Regulations. The NSW CoP for Strata control in underground coal mines runs to 102 pages and specifically addresses how to meet regulatory requirements (NSW Govt. 2015). The Australian CoP for Ventilation of Underground Mines was drafted in 2011 by Safe Work Australia but does not appear to have progressed since the draft stage. (Safe Work Australia 2011)

The first step in developing a TS must be to assemble a group of individuals with the desire to develop this project further. Ideally, the widest possible representation from industry, consulting, suppliers and academia should be sought with the aim of assembling an expert panel that has the knowledge and expertise to develop the proposed Technical Specification.

Standards committees are generally run on a voluntary basis, with only a part-time project manager assigned by Standards Australia to help manage the processes but not to contribute directly to production of the standard or TS. Under SA procedures, a TS is prepared by an expert Working Group, which is a sub-set of an existing Standards Technical Committee. Thus the underground coal mining industry would need to find a sponsor this particular TS and to overview the publication of the TS.

Ideally, some members of the initial interest group will have had previous experience on Code committees or contacts on existing committees that could be useful in finding a suitable structural committee to oversee production of a Technical Specification for Structural Design of VCDs.

The list of issues that should be addressed by a Code of Practice can be made as short or as long as the Committee decides it should be. In practice, structural design Standards vary from thick documents covering all aspects of detailed design through to slim manuals providing basic information to experienced practitioners. One of the first tasks for a Working Group will be to decide what level of detail will be included in the document to be produced.
CONCLUSION

It is critically important to maintain an effective, safe and efficient ventilation system in an underground coal mine and ventilation control devices (VCDs) play an essential role in mine operations and safety. It is therefore concerning that no agreed or accepted standard exists for the structural design of VCDs. With no required design standard in place, design of VCDs in Australia is currently performed in accordance with individual engineer's understandings and preferences. The result is that VCDs that are designed for identical applications can vary widely in their construction details. At best, this represents wasted resources where VCDs are over-designed and at worst underdesigned VCDs will fail when they are exposed to explosion or water pressures. The effectiveness of a VCD to perform its function is dependent on a variety of factors, only one of which is design. These factors can be summarised as follows:

- Geology
- Siting
- Design
- Materials
- Construction
- Inspection and Testing
- Maintenance

It is proposed that a Technical Specification for Structural Design of VCDs should be developed by the industry to address all of the above issues. Future review of the Technical Specification can be undertaken so that new developments in technology can be shared within the industry. This will provide a higher level of safety to underground miners by minimising the risk of VCD failures.

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